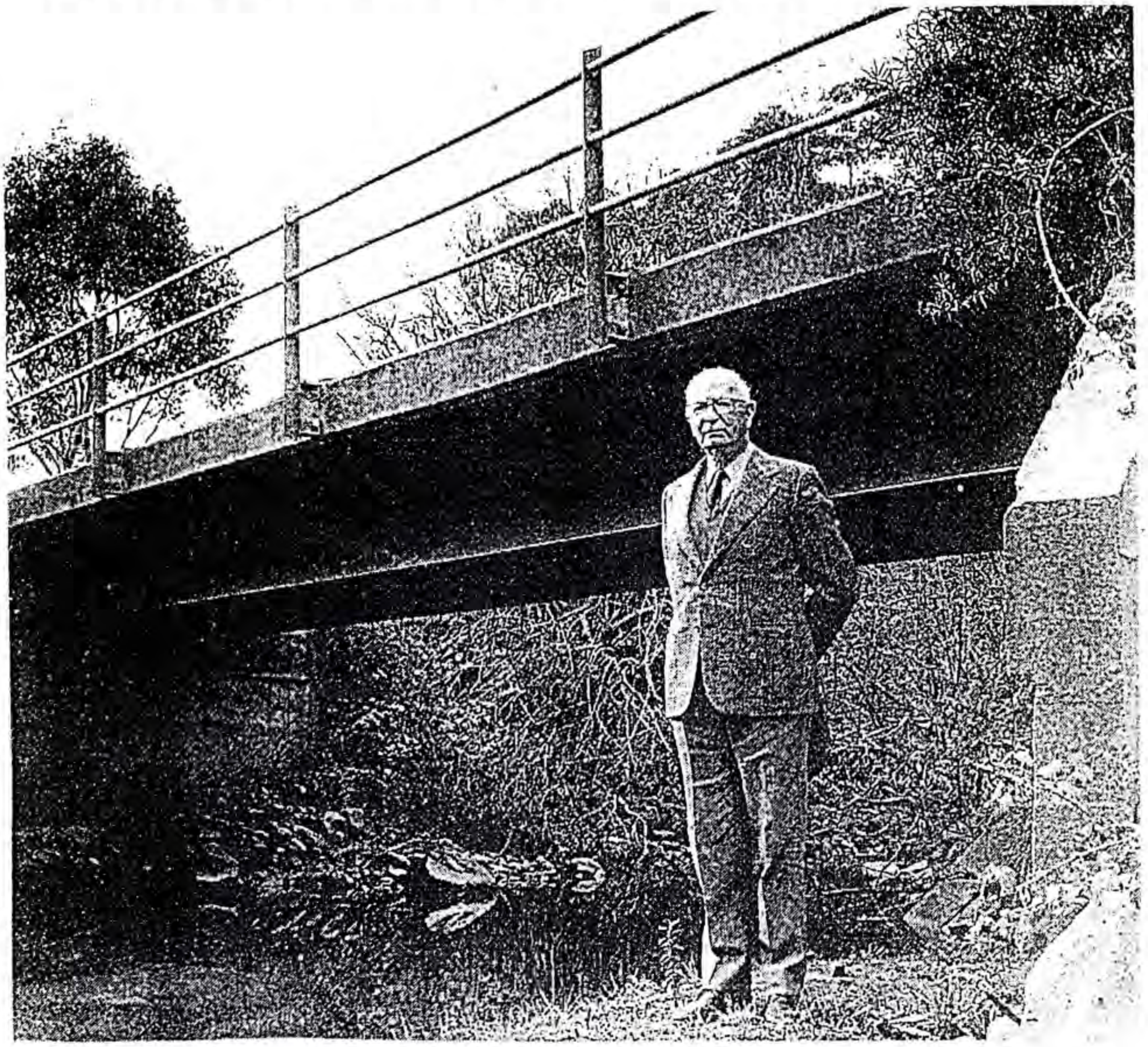
VINCENTS RIVULET BRIDGE



A Tribute to the late Sir Allan Walton Knight, Kt., CMG.

Submission for an
Historic Engineering Marker
from
The Engineering Heritage Committee
Tasmania Division
Institution of Engineers, Australia
1999
and Plaquing Ceremony Report

CONTENTS

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VINCENTS RIVULET BRIDGE

A Small Bridge with an Interesting History

Close by the Southern Outlet Road that leads from Hobart City to Kingston and the southern suburbs is a small occasionally used rural road, Proctors Road. Proctors Road crosses Vincents Rivulet and it was at this crossing in the early 1930's that a small bridge was proposed incorporating pioneer ideas that were later to receive world-wide recognition and adoption. As far as can be determined, this bridge is the first composite concrete deck/steel girder bridge built in Australia and perhaps in the world.

The story of the bridge and how it came to be constructed is an interesting one. Mr. Allan Knight (later to be Sir Allan Knight, Commissioner, Hydro-Electric Commission, Tasmania) was working as a part-time demonstrator in the Engineering Department, University of Tasmania, under the direction of Professor Alan Burn. The Public Works Department approached Professor Burn in early 1932 to see whether he could assist them in determining the distribution of wheel loads on a concrete deck to the supporting steel beams. Professor Burn accepted the commission and asked Mr. Knight if he would undertake the work. Mr. Knight thought that the best way to investigate the problem was to build a structural model and apply known point loads at various positions on the deck and to measure deflections by means of an extensometer. He applied the point loads and measured deformations and during the course of the investigation noted that, on inspecting the underside of the model, the bakelite deck had actually lifted off and parted company with some of the supporting beams. He realised that separation did not take place on actual structures and to overcome this difficulty he melted paraffin wax along the top of the timber beams and stuck the bakelite deck to the timber beams with paraffin wax. He then resumed his testing of the model and noticed that the deflections now being given were much smaller than previously indicated, and that by gluing the deck to the beams he had produced a stiffer deck system. He therefore concluded that if a concrete deck could be made to act with the supporting steel beams without slippage or separation, economies could be afforded in bridge decks for given loadings.

In September 1932, Mr, Knight joined the staff of the Public Works Department in their Bridge Section and proposed that the Department adopt the principle in bridgeworks of making the concrete deck act with the supporting steel beams. The Department's Director, Mr. G.D. Balsille, was interested in promoting the proposal and, after some initial considerations of building a concrete/steel structural model, actually decided to proceed and build a full scale bridge to test out the system. In order to guard against an unsatisfactory performance, a quiet back road was selected so that if problems arose no serious dislocation of traffic would occur. Composite action between the concrete deck slab and the steel beams was to be achieved by welding round bar stirrups to the beams and turning them up into the bridge deck. The stirrups were placed in pairs at 7" centres, and varied in diameter from 3/8" at the centre to 5/8" at the ends of the beams. In order that the composite section should carry both dead and live load, the beams were propped until the concrete of the deck slab has set and gained strength. The layout of beam and deck slab is shown on the accompanying figure. (Fig 1)

2/...

The bridge was a single lane structure 34' - 0" overall length, 12' - 0" overall width and 10' - 8" between kerbs. The bridge upon completion was test loaded with a 10.6 tonnes truck loaded with road metal and came though with flying colours. Messrs. Balsille and Knight were in the group that had gathered to witness the test loading and celebrated their success in an appropriate manner after they had left the site. The success of the Vincents Rivulet Bridge, in demonstrating the validity and the value of composite action between deck and supporting beams, encouraged the wider and further use of composite action in Departmental bridgeworks. Mr. Knight published a Paper on composite action in "The Commonwealth Engineer" (April 1st, 1933 issue) and awakened wide interest throughout Australia, with other States adopting the principle and constructing bridgeworks incorporating composite action.

The bridge is being maintained in first class condition in keeping with its historic nature and will continue to enjoy a high place of honour amongst the many bridges in the State of Tasmania. The photograph shows the bridge as it is today, on its very quiet country road, but now immediately adjacent to Hobart's busy Southern Outlet Road.

<u>IVAN GAGGIN, MIEAust CPEng.</u>

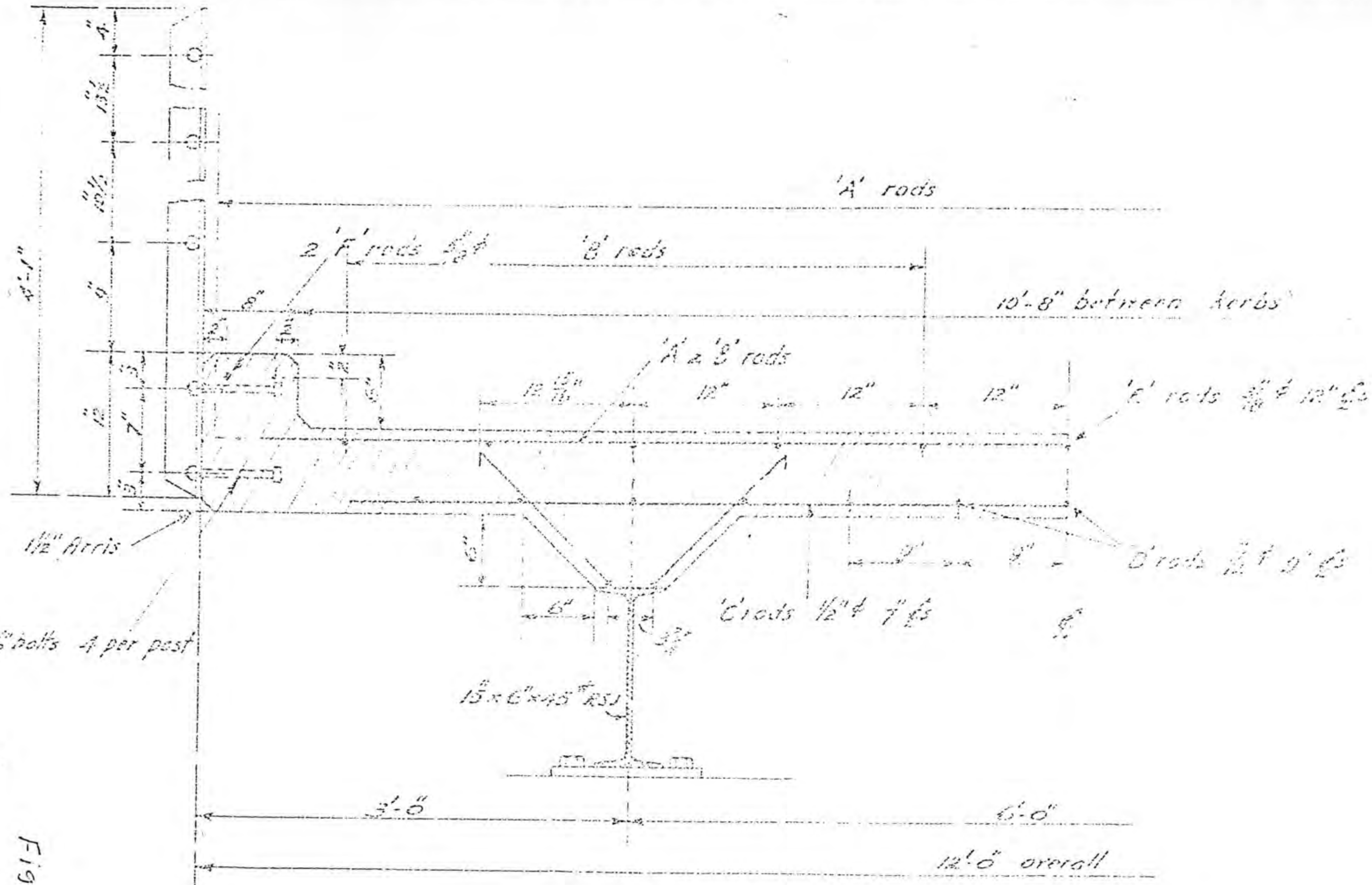

Fig 1

SECTIONAL ELEVATION

Bridge
HOBART
Glenorchy
Gagebrook
Old Beach
Otago
MT DIRECTION
Risdon Vale
Lindisfarne
Montagu Bay
Rosny
Bellerive
Howrah
Mornington
Warrane
Rokeby
Tranmere
Cambridge
Cambridge Aerodrome
TASMAN HIGHWAY
PITT WATER
RIVER DERWENT
RALPHS BAY
MEEHAN RANGE
GRASSTREE HILL
Grasstree Hill
Craigow Hill
Mt Stuart
Knocklofty
West Hobart
North Hobart
South Hobart
New Town
Moonah
West Moonah
Lenah Valley
Sandy Bay
Lower Sandy Bay
Wrest Point
Battery Point
Glebe
Macquarie Point
Mt Nelson
MT NELSON
Taroona
Bonnet Hill
Kingston
Kingston Beach
Blackmans Bay
Ridgeway
Fern Tree
Chimney Pot Hill
Lauderdale
Sandford
Mt Mather
Beauvais Hill
Kilgaran Hill
Gravel Hill
Mt Augustus
Mortimer Bay
Opossum Bay
Droughty Hill
Gellibrand Point
The Spit
MT RUMNEY
Single Hill
Racecourse Flats
Acton
Mt Lord
Richmond Park
Lowlands
Cold Blow
Strathayr
Glenayr
Karls Hill
Downhams Hill
Gunners Quoin
Madmans Hill
Blacksmiths Hill
Nichols Hill
Clemens Hill
Eagle Hill
Caves Hill
Seagers Hill
Sugarloaf Hill
Rocky Tom
Flagstaff Hill
Simmons Hill
Golden Hill
Breakneck Hill
Canopus Hill
Tunnel Hill
Knopwood Hill
Mornington Hill
Glebe Hill
Skillion Hill
Clarendon Vale
Stanfields Hill
Guy Fawkes Hill
Acton Hill
Barilla Bay
Shark Point
Flexmore Park
Greenfields
Uplands
Rosetta
Montrose
Derwent Park
Lutana
Cornelian Bay
Elwick Bay
Prince of Wales B
Goodwood
Risdon
Cleburne Pt
Bowen Park
Dowsings Pt
Crayfish Point
Cartwright Point
Tyndall Beach
Alum Cliffs
Bonnet Point
White Rock Point
Icehouse Bluff
Mary Ann Bay
Droughty Point
Trywork Point
Gibsons Point
Tranmere Point
Dixon Point
Mario Point
Kangaroo Bluff
Second Bluff
Howrah Point
Punchs Reef
Sandy Bay Point
Little Sandy Bay
Blinking Billy Point
Secheron Point
Sullivans Cove
Wharves
Summerleas
Picket Hill
Doctors Hill
Spring Farm
Maranoa Heights
Kingston Heights
Beronia Point

STATEMENT OF SIGNIFICANCE

The Vincents Rivulet Bridge was built to prove the practicality of A.W. Knight's Composite Beam Theory.

Composite action enabled the concrete deck to be loaded completely in compression and the steel beams completely in tension, stress conditions in which the strength of each material was fully utilised.

The bridge not only performed under load as expected but demonstrated that considerable cost savings in both steel and concrete could be achieved in this type of bridge. (Ref 4)

The next step was to build a larger bridge of the same type, and in 1933-34 a 7 span composite beam bridge 430ft in length 23ft wide was built over the Leven River at Ulverstone in Northern Tasmania. It remains in daily use. The Leven River bridge was designed by A.W. Knight who also supervised the construction.

This was then followed by many similar bridges throughout Australia and overseas.

Commemorative Plaque Nomination Form

Date 24 - 6 - 99

To:
Commemorative Plaque Sub-Committee
The Institution of Engineers, Australia
Engineering House
11 National Circuit
BARTON ACT 2600

From Engineering
Heritage Committee
Tas Div I.E. Aust.
Nominating Body

The following work is nominated for a:-

- ~~National Engineering Landmark~~
- Historic Engineering Marker
*(delete as appropriate)

Name of work VINCENTS RIVULET BRIDGE

Location, including address and map grid reference if a fixed work
On Proctor's Rd between Hobart and Kingston
Map Derwent 8312 Co-ords EN 253440

Owner Department of Transport Tasmania

The owner has been advised of the nomination of the work and has indictated (attach a copy of letter if available)
Copy attached

Access to site Via Proctor's Rd

Future care and maintenance of the work Is and will continue to be maintained by the Dept of Transport

Name of sponsor I.E. Aust and the Royal Society of Tasmania

For a NEL, is an information plaque required?

Chairperson of Nominating Committee

R. L. Bennett
Chairperson of Division Heritage Committee/Panel

ADDITIONAL SUPPORTING INFORMATION

Name of work....VINCENTS RIVULET BRIDGE....

Year of construction or manufacture....1932....

Period of operation....1932 to date....

Physical condition....Very Good....

Engineering Heritage Significance:-

Technological/scientific value..Prototype Composite Beam Bridge

Historical value First application in Australia of Composite Concrete/Steel Beam Theory

Social value. Reduced cost of future concrete bridges.....

Landscape or townscape value....Nil....

Rarity....

Representativeness. First of its kind in Australia....

Contribution to the nation or region. Of National significance in Bridge design and construction

Contribution of engineering. Proved the method for larger bridges of this type

Persons associated with the work (Sir) A.W. Knight, Prof. A. Burn, G.D. Balsille

Integrity....Remains as built....

Authenticity....Original....

Comparable works(a) in Australia....None at that time....

(b) overseas....Unknown....

Statement of significance, its location in the supporting documentation......

....See attached statement....

Citation (70 words is optimum)....

....See attached proposed wording....

....

....

....

Attachments to submission (if any) Eng. Conf. paper No. 970 by A.W. Knight 1934

Proposed location of plaque (if not at site)....

UNDER ROYAL PATRONAGE SINCE 1844

THE ROYAL SOCIETY OF TASMANIA

1843

Mr. David Freestun.
26. Kenton Road.
Geilston Bay. 7015.

Dear David,

The matter we discussed in relation to possible plaquing was discussed at our recent Council Meeting. I was asked to convey to your organisation, and you in particular, that we would like to progress the idea further. I look forward therefore to hear how your committee felt about the idea. When they have met I look forward to hearing from you.

Yours Sincerely,

Michael

Michael Readett.
Hon. Secretary.

10.6.97.

THE TASMANIAN SOCIETY FOR THE ADVANCEMENT OF KNOWLEDGE

9

19th June, 1997.

Dr. M. Readett,
Honorary Secretary,
The Royal Society of Tasmania,
G P O Box 1166M,
HOBART TAS 7001

Dear Dr. Readett,

COMPOSITE BRIDGE - PROCTORS ROAD

I refer to your letter of 10th June 1997 to David Freestun and advise that this Committee is keen to proceed with plaquing the bridge and will arrange for the necessary submission to be forwarded to the Institution of Engineers.

The process will require some months and we will contact you immediately prior to the submission of the documents to arrange allocation of any costs involved.

Yours sincerely,

HARRY deV GILBERT, MIEAust., CPEng.,
Assistant Secretary,
Heritage Committee, Tasmania Division

Mr. H. deV. Gilbert. MIEAust.,CPEng.,
Assistant Secretary,
Heritage Committee,
Tasmanian Division.

Dear Mr. Gilbert,

Thank you for your letter about the Composite Bridge in Proctors Road. The Royal Society is delighted with your news. We look forward to hearing from you again in due course.

Yours Sincerely,
Michael Readett
Michael Readett.
Hon. Secretary.

21.6.97.

11

Tasmania

DEPARTMENT of
TRANSPORT

Enquiries: R W McGee
Phone: (03) 6233 3514
Fax: (03) 6233 6657
Email: rw-mcgee@dot.tas.gov.au
Your Ref:
Our Ref: 004089

29 May, 1998

Mr Keith Drewitt
Chairman, Engineering Heritage Committee
The Institution of Engineers Australia
2 Davey Street
HOBART TAS 7000

Dear Keith

HISTORIC ENGINEERING MARKER - VINCENTS RIVULET BRIDGE

Thank you for your letter of 28 May regarding the proposal for the Royal Society of Tasmania and the Institution of Engineers Australia to locate an Historic Engineering Marker on or near the bridge.

The Department has no objection to the placement of such a marker. Given the form of the bridge and the need to ensure that the bridge can be readily maintained in the future, it is likely that a location near the bridge would be more appropriate. Would you please provide details of the selected location before proceeding with installation.

Yours sincerely

Rod W McGee
MANAGER ASSET STRATEGIES

A tribute to the late Sir Allan Walton Knight Kt CMG

Allan Walton Knight who died at Hobart on 14 May 1998 was born at Launceston on 26 February 1910.

He was undoubtedly one of Australia's most outstanding engineers having a consistent mixture of personal excellence and practical achievements. Together with a long and dedicated commitment to Tasmania including 10 years as Chief Engineer in the PWD and 30 years as Commissioner and Chief Executive of the Hydro Electric Commission.

As a junior, Knight was appointed to assist Professor Alan Burn, Professor of Engineering at the University of Tasmania under whose guidance he modelled and successfully proved the Composite Beam Theory which was subsequently adopted world wide.

To prove the practicability of the theory the Director of PWD authorised the building in 1932 of an 11 metre long prototype Composite Beam bridge near Hobart on Proctors Road over Vincents Rivulet which was test loaded with a truck carrying 7.9 tons on the rear axle and 2.7 tons on the front axle giving a deflection of only 0.06 inches.

The method was subsequently applied in 1933-34 to the building of a bridge over the Leven River at Ulverstone with 7 spans of 18.6m and width of 8.9m which is still in use today. A.W. Knight designed the bridge and supervised its construction.

Reference: Australian Register of Historic Bridges by Professor O'Connor.

A Tribute to the late Sir Allan Walton Knight, Kt., CMG.

Allan Walton Knight, who died at Hobart on 14 May 1998 was born at Launceston 26 February 1910, son of the late G. W. Knight and served Tasmania as undoubtedly one of Australia's most outstanding engineers. He had a strong and consistent mixture of personal excellence and practical achievements together with a long and dedicated commitment to Tasmania including 30 years as Commissioner and Chief Executive of the Hydro-Electric Commission (7 August 1946-28 February 1977).

In all this he was strongly supported by Lady Knight (Margaret, daughter of the late H. Buchanan MBE.) and their children, Charles, Helen and Tim. Margaret's cheerful approach to the staff and the construction workforce endeared her to them and their families.

Allan's family background was in farming and education. His relative, William Corin who was appointed engineer-in-charge of The Duck Reach Hydro-Electric power development in October 1895 and successfully developed its then small direct-current supply to a much larger output with 3 phase alternating current machines before leaving for NSW in 1908 where he was appointed chief engineer and adviser to several governments. He together with Sir John Butters (Great Lake and the Waddamana Hydro-Electric power development) and Knight formed the strong triangular base upon which the Hydro-Electric Commission's installed capacity grew from 172 to 1515 mega Watts and its workforce from 1000 to more than 5000 during his direction of the Commissioner.

Whilst employed by the Education Department, Knight was appointed Assistant to Professor Alan Burn, Professor of Engineering at the University of Tasmania under whose guidance he modelled and successfully proved the Composite Beam Theory which was quickly adopted world wide.

This showed that major savings could be obtained by joining the steel tension members of for example the steel girders of a bridge physically and dynamically to the concrete deck, converting what was otherwise dead load into the compression member of the Composite Beam.
With the approval of George Balsille, the Director of the Public Works Department and their Minister, a prototype bridge 11 metres long was built at Proctors Road, shown in the photograph above - the Minister requesting that its site not be too obvious in case of failure.
Knight stayed under the completed bridge to measure its deflections under the test loading, static and impact created by large trucks loaded with rock passing over it. The deflections measured were well within the safety factors required and Knight's confidence and safety was proved.
Subsequently in 1932 he was appointed a PWD engineer where his design and drafting skills were outstanding and highly regarded as was his innovative approach. He carried out many designs and their estimated costs at home or subsequently when appointed resident or site engineer at Ulverstone and Scamander in his office shed at site works in the evenings.
In his own time he produced a composite beam design in the mid 1930's for the proposed new road bridge over the Leven River at Ulverstone. On the 28th June 1934, the Director wrote "Mr. Knight is in charge of the construction of Leven Bridge, Ulverstone for £13,700". "He also he has shown conspicuous ability in design research and organization of works".
His estimates indicated a 10% reduction in cost and reduced construction time. When the Minister of Works the Hon. Major T. H. Davies, MC., DSO., RE., approved Balsille sent Knight to build it where he introduced temporary toms to prestress the concrete and further reduce its cost.
Similarly his proposals for the fully welded steel girder bridge at Scamander, the third earliest in Australia, earned him the task of supervising its construction and again he spent most of his evenings on site designing and providing drawings for its completion. Although presently bypassed this bridge deserves preservation as a historic memorial to the skills and dedication of Knight and his fellow engineers in the Public Works Department as a major advance in engineering technology.
(28) In 1937 at 28 years of age he was appointed to the newly created position of Chief Engineer of the Public Works Department being promoted over several older and more senior engineers, one of whom, Robert Sharp later Director, told the writer that it was "completely acceptable .... he was so outstandingly competent".
His early interest is indicated when he convinced his father that he should withdraw from Hutchins School after attending on a scholarship for a few weeks because they did not have classes in drawing or drafting and he attended the Hobart Technical School instead where in 1929 he qualified for a Diploma of Applied Science. His subsequent qualifications were obtained at the University of Tasmania; graduating Batchelor of Science (1932), Batchelor of Engineering (1932), Master of Engineering (1935) and Batchelor of Commerce (1946). In 1989 the University of Tasmania conferred on him the Honorary Degree of Doctor of Engineering.
Resulting from a programme of major bridge construction, Knight was sent to America and Canada to examine advanced bridge design and construction. It is typical of his professionalism that he presented his Minister with a completed 121 page booklet of his experiences and conclusions within a few weeks of returning to Hobart. Part III of which is an excellent description of roadworks and their contribution to scenic and tourist values with forest and landscape management.
Resulting from his recommendations and imput the Australian first all-welded railway bridge was built across the Derwent River at Bridgewater. Subsequently he designed and held patent rights to the floating bridge across the Derwent River Estuary at Hobart. In both these cases an experienced welding consultant, David Isaacs of Melbourne was engaged to detail and advise on the steel structures, particularly the navigational lift spans and the design, fabrication and testing of their gusset plates and junctions. He shared equally with Knight the considerable bonus presented by the owners of the Floating Bridge.

Knight was presented with many professional awards and honours, Fellow of the Institution of Engineers, Aust. (1934), Honorary Fellow (1983), Warren Memorial Prize (1934), Peter Nicol Russell Medal (1963), William Charles Kernot Medal (1963), Chapman Award (1974) and the John Storey Medal (1975). He served on many Councils of professional nature including the College of Advanced Education, University Council, C.S.I.R.O., Churchill Trust, Scenery Preservation Board (35 years) and the Water Research Foundation.

He was also appointed Commissioner of the Joint Tasman Bridge Restoration Committee, responsible for directing and managing its rebuilding where required, including widening. Also on the selection authority for the form and site of the Bowen Bridge upstream. He served for many years on the Australian Universities Commission, Councillor Standards Association and the Electric Supply Association of Australia.

He was the first Tasmanian invited to present the Sir John Morris Memorial Lecture (1966) joining such outstanding Australians as Sir Owen Dixon (1958). His well researched "The Quest for Power" was preprinted in a booklet of 25 pages.

His contribution to engineering education, practise, management and leadership has made him one of the most outstanding Australians of this century.

"Work was my hobby, I enjoyed it and never found it any sort of a burden".

He well understood John Donne's "No man is an island sufficient unto itself.........." and was supported by a dedicated and highly committed staff which included some 300 professional engineers, geologists, surveyors, accountants and economists, many engineers having a Batchelor of Economics second degree. They included ~~two~~ three Rhode Scholars, a Harkness Fellow et alia. In tunnel boring the Hydro Electric Commission set world records which have not been surpassed, led world-wide practise in large rockfill dams with upstream concrete membranes and at some 2800 ft. static head-built the worlds 6th highest head hydro-electric power station at Poatina. All of which contributed to maintaining the low cost of Tasmania's hydro-electric power.

The large field construction work force included many migrants seeking a new life in a new country, mainly employed on day-labour conditions. Some of their families worked for the Hydro-Electric Commission for three generations.

All proposals for increases in the system's capacity were compared and costed with the alternatives available; submitted to the Minister, publicity debated and approved by both Houses of the Tasmanian Parliament. It was always clear that thermal including atomic power would be much more expensive than hydro-electric power.

The strong emphasis on cost and budget control was emphasised by Sir Allan's definitive paper "The Design and Construction of Composite Slab and Girder Bridges", IE Aust Transactions 1934. It states p21 "The 34 ft. single span experimental bridge (Proctors Road - see photo) gave savings of 29.5% and 35% on the steelwork of the superstructure of the 427 ft. bridge of 7 spans.

The professional engineer goes back to the beginnings of civilisation, an example of which is the appointment of Sextus Julius (b.AD 35) as Curator Aquarium for Rome. He was then Governor of Britain (AD 76) and had studied at Alexandria and of course the Popes are engineers, Pontiff, the bridge builder (between Earth and Heaven).

His sporting activities were of equal standard to his technical abilities. He rowed and played tennis for his University and was Tasmanian open singles tennis champion for a number of years, also Royal Tennis Champion 11 times and Australian Champion three times. He represented Tasmania in the Linton Cup.

In World War II Knight resigned to enlist in the Forces but his resignation was refused and his offer to serve was restricted. Thereafter he handled two almost full time jobs by adding the Assistant Director of Works, Tasmanian Command from 1939 to 1940 and Major commanding the Royal Australian Engineers (Tasmanian Command) 1940-42 to his civilian work load.

He was Chairman of the Tasmania Division Council of the Institution of Engineers, Australia and a Federal Councillor (1942).

As confirmed by his above listed activities he published and presented professional papers to the Institution of Engineers Australia and other learned institutions, conferences et alia.
In retirement he established a reputation as a water colourist and enjoyed wood turning evidence of which maybe seen in the Knight/Warner Perpetual Trophy of Huon Pine in the form of a pedestal fruit compote for the winner of the Inter Club Tennis competition with Athenaeum Club (presently held by the Athenaeum Club) and in the Huon Pine cap atop the white post at the foot of the back steps of the Club.
Sir Allan was elected by the members nemine contradicente a Life Member of the Tasmanian Club in October 1997.

Vale Allan Walton Knight - a great Tasmanian.

Henry H. McFie MC., F.IE Aust.

6/6/98

Suggested Plaque Wording

HISTORIC ENGINEERING MARKER

VINCENTS RIVULET BRIDGE

THIS IS THE FIRST COMPOSITE BEAM BRIDGE IN AUSTRALIA. THE BRIDGE WAS DESIGNED BY A W KNIGHT AND BUILT BY THE PUBLIC WORKS DEPARTMENT IN 1932. LOAD TESTING OF THE BRIDGE CONFIRMED HIS THEORY THAT THE CONCRETE DECK AND STEEL GIRDERS WOULD CARRY A MUCH HEAVIER LOAD IF RIGIDLY JOINED TOGETHER. THE METHOD WAS THEN ADOPTED FOR MUCH LARGER BRIDGES BOTH IN TASMANIA AND IN OTHER STATES, WITH SIGNIFICANT COST SAVINGS.

DEDICATED BY

The Institution of Engineers, Australia

and

The Royal Society of Tasmania

1999

VINCENTS RIVULET BRIDGE

Photo 1 Vincents Rivulet Bridge 1990

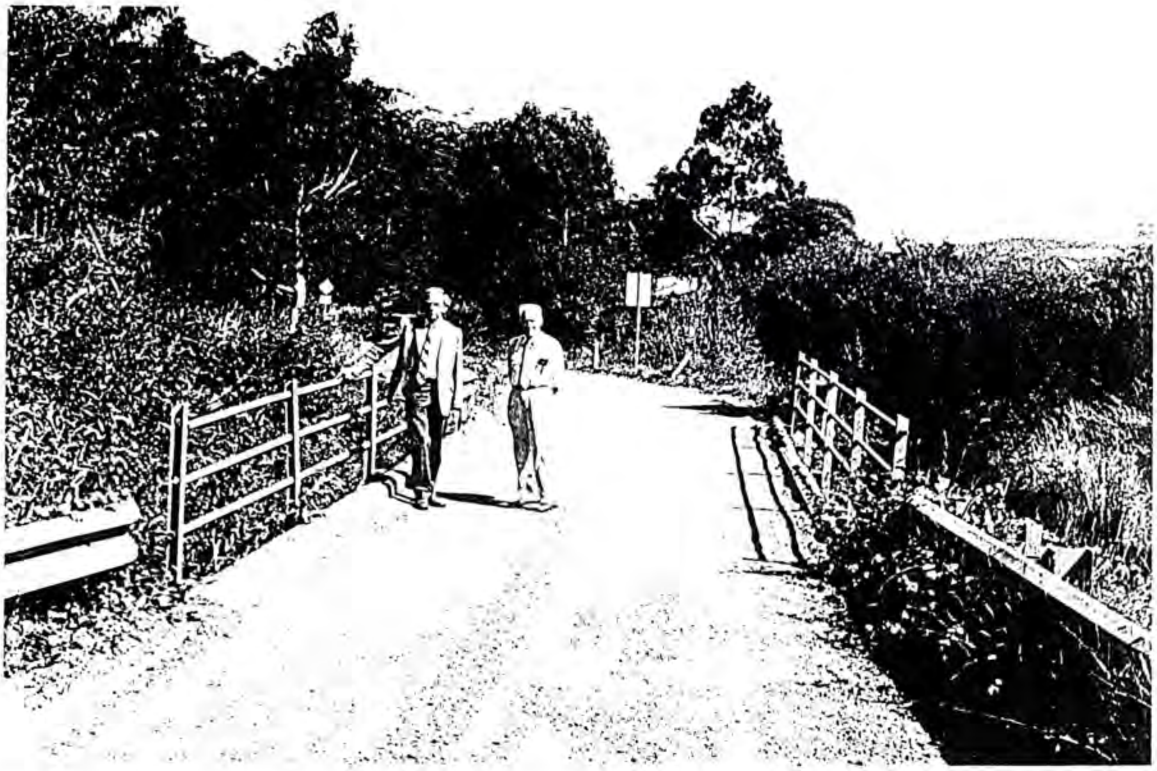

Photo 2 Proctors Rd Crossing Vincents Rvt. Bridge
K. Drewitt and H. McFie 1990

References

Ref 1: Composite Bridge Under Threat
Engineers Australia 14/6/85

Ref 2: General Arrangement Drawing
P.W.D. 23/11/32

Not scanned
Too big

Ref 3: A New Type of Road Bridge
by A.W. Knight BSc., B.E., Jr. IEAust.
The Commonwealth Engineer April 1933 p. 267

Ref 4: The Design and Construction of Composite Slab and Girder Bridges
by A.W. Knight BSc., B.E.
IEAust Transactions Vol XV p.10 Jan 1937

Ref 5: Register of Australian Historic Bridges
by Prof. C. O'Connor p.57 1983

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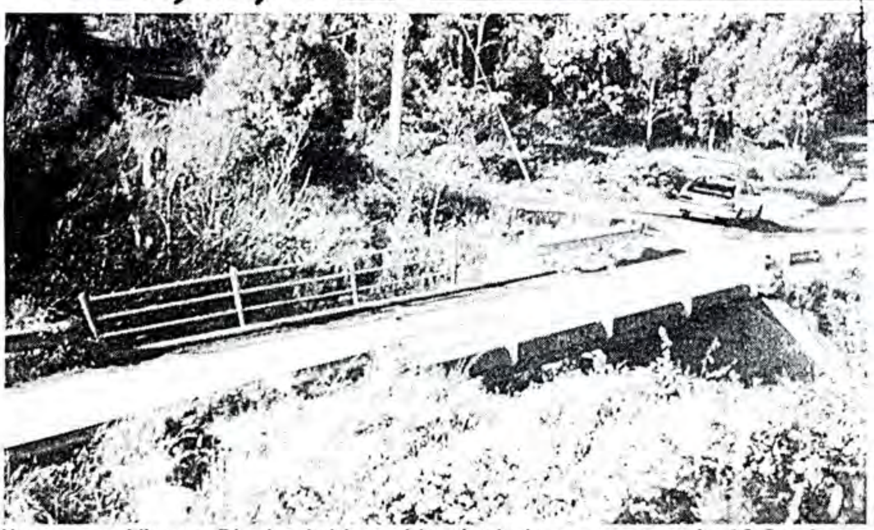
Composite bridge under threat



Known as Vinces Rivulet bridge, this single-lane structure is 10.3m long, 3.6m wide and 3m between kerbs. It is maintained today by the DMR.

An insignificant-looking bridge on a dirt road which crosses a creek just outside Hobart may have been a world first in engineering terms.

Department of Main Roads, Tasmania, division engineer, bridges Ivan Gaggin believes this bridge to be the first composite concrete deck/steel girder bridge built in Australia and perhaps the world.

But the bridge is under a threat. The road it is on runs parallel with the Southern Outlet, a modern major road link between Hobart and Kingston.

Long-term plans for this road include a further upgrading which could involve widening and absorbing the side road and its bridge.

Although roadworks are unlikely to start before 1987 at the earliest, Gaggin is keen to confirm the bridge's pedigree and, if it checks out as expected, to preserve at least the deck.

How the bridge came to be built and why it was erected on a side road has been researched by Gaggin who says Alan Knight, later HEC commissioner Sir Alan Knight, built it for the Public Works Department to help determine the distribution of wheel loads on a concrete deck to the supporting steel beams.

Knight built a model using timber beams, and a bakelite deck and applied point loads and measured deformation. He noticed, on looking at the underside of the model, that the deck had lifted off some of the supporting beams.

Knight realised that separation did not take place on actual structures and to overcome this he melted paraffin wax along the top of the timber beams and stuck the deck to the beams with wax.

Further testing showed deflections were now much smaller and that the glueing had produced a stiffer deck.

His conclusion was that if a concrete deck could be made to act with the supporting steel beams without slippage or separation, a more economical bridge deck could be built for a given loading.

When Knight joined the PWD in 1932 he built the existing bridge on a side road so that if there were problems with it there would be no traffic disruption.

Composite action between the concrete deck slab and the steel beams was achieved by welding round-bar stirrups to the beam and turning them up into the bridge deck.

Immediately after its construction the bridge was tested by running a truck loaded with road metal across it. □

A New Type of Road Bridge

By A. W. Knight, B.Sc., B.E., Jr.I.E.Aust.*

An economical form of construction for short span road bridges consists of a concrete slab deck on steel joists, the main advantage as compared with reinforced concrete tee beam construction being in the reduction of falsework, since the shuttering for the deck slab can be carried directly on the joists. Up to the present it has been necessary in the design of such bridges to provide steel joists sufficiently strong to carry the whole of the longitudinal bending moment, as the adhesion between the concrete and the steel cannot be relied on to provide any appreciable tee beam action.

The results of a simple experiment illustrating such tee beam action are shown in Fig. 1. Two light wooden laths were placed in contact in such a way as to represent respectively the flange and web of a tee beam, and deflection measurements were made for varying concentrated loads applied at mid span. A second series of deflections was obtained after the laths had been joined by a film of paraffin wax 1/16 in. in thickness. The graph shows the increase in strength of the arrangement due to the introduction of the wax. Breakdown of the tee beam action of the laths occurred at a load of 350 lb. owing to failure of the wax. The shear stress in the wax at the point of failure was 137 lb. per sq. in.

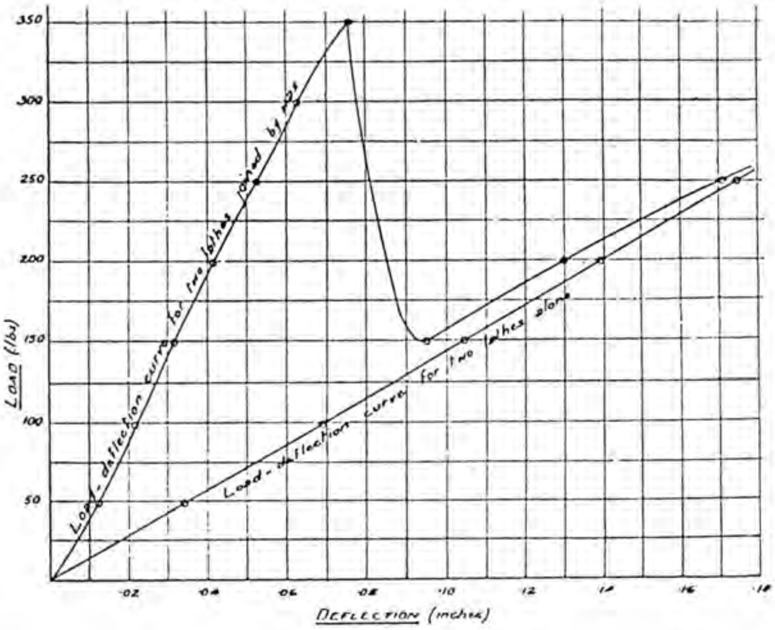


Fig. 1

In order to economise in steel and to enable the steel joist and slab deck type of construction to be used for longer spans, the public works department of Tasmania has recently developed a design in which the wax of the above experiment is replaced by steel shear reinforcement welded to the top of the joist and embedded in the concrete slab. Preliminary tests made on a small model at the university of Tasmania indicated that the expected tee beam action could be fully developed with a comparatively small amount of shear reinforcement. The model consisted of a 5-in. x 2½-in. x 15-lb. steel joist carrying a 9-in. x 3-in. concrete slab reinforced for shear. The reinforcement was provided by ¼-in. diameter round steel bars welded to the top of the joist and distributed in the necessary manner to carry the shear forces in the slab. The test piece was designed to carry an additional load of 61.2 per cent. over that for the joist above; the increase in load obtained under test was 63.2 per cent.

Preliminary tests having shown such satisfactory results, the 34-ft. span single-track bridge shown in Fig. 2 has been designed and built using the same principles at a cost of £3/10/- per ft. run for the super-structure, and £5/10/- per foot run for the sub-structure, a total of £9 per ft. The saving in the cost of steel work amounted to 29.5 per cent., but as

*Dept. of Public Works, Tasmania.

the bridge was designed to test the technical principle rather than the economics of such a structure this figure could possibly be improved upon. A large saving would result where the adoption of this form of construction saved either a pier or the use of a truss or plate girder.

The new bridge replaces an old wooden structure, and is designed to allow rubbish carried down by flood waters, which periodically inundate the river flat, to pass through the fence without undue obstruction. The bridge consists of a 7-in. deck slab 10 ft. 8 in. between kerbs, supported by two 15-in. x 6-in. x 45-lb. steel joists at 6 ft. centres. Concrete haunches 6 in. deep with sides sloping at 45 deg. were cast between the slab and the top flanges of the joists. A cross section is shown in Fig. 3. The bridge was designed to carry a live load of 10 tons on two axles at 14 ft. 6 in. centres with 7 tons 6 cwt. on one axle. The shear reinforcing rods were placed in pairs at 7 in. spacing along the joists. The rods varied in diameter from $\frac{3}{8}$-in. at the centre to $\frac{5}{8}$-in. at the ends of the joists. A general idea of the arrangement of the reinforcement can be gained from Fig. 4.

Fig. 2—Bridge of 34 ft. Span on Proctor's Road, about eight miles from Hobart

In order that the composite section should carry both dead and live load, the joists were propped until the concrete of the deck slab had set. To obtain a better stress distribution in the steel an initial camber was placed in the beams by bolting the ends down on to the abutments over the central prop. The level of the prop was fixed to give the required stress distribution in the steel. The arrangement of the props can also be seen in Fig. 4. The final stress distribution under dead load and live load is shown in Fig. 5. It will be seen from this stress diagram that not only is the whole of the steel in tension but the tension is greater at the top flange than at the bottom. This simply indicates that the camber is not entirely removed even under maximum live load.

A Leyland truck, with 7.9 tons on the back axle and 2.72 tons on the front axle was used as a live load in a test carried out on the bridge. A central deflection of 0.110 in. occurred on removal of the props as a result of a bending moment of 63.6 ft.-tons due to dead load. A

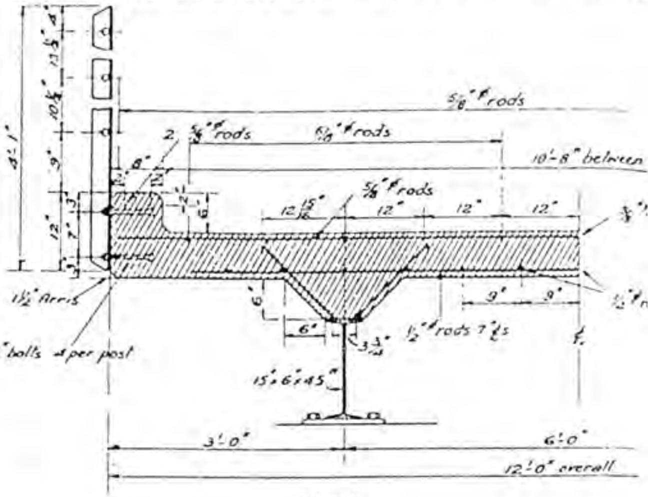


Fig. 3

central deflection of 0.059 in. was obtained with the above live load which produced a bending moment of 33.1 ft.-tons (per joist). By placing the same live load on the extreme edge of the bridge the maximum deflections of the two joists were 0.081 in. and 0.053 in. These values were

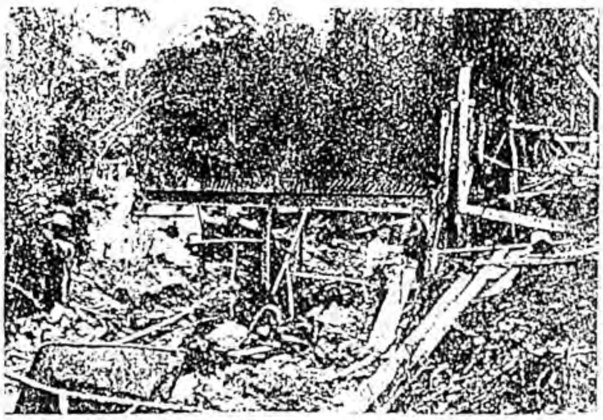


Fig 4—Arrangement of Shear Reinforcement

exactly reversed when the load was placed on the opposite side. It is difficult to understand why the sum of these latter deflections does not approximate more closely to the sum of the deflections with the original symmetrical loading. Assuming the load statically applied in the eccentric position (i.e., distributed between the two beams in accordance with statical principles) the calculated deflection values are 0.103 in. and 0.031 in. which gives some indication of the extent of the load distributing effect of the concrete slab.

The above figures for deflection were only obtained after a number of applications of the load, as an initial tendency towards permanent set and gradual increase and decrease of deflec-

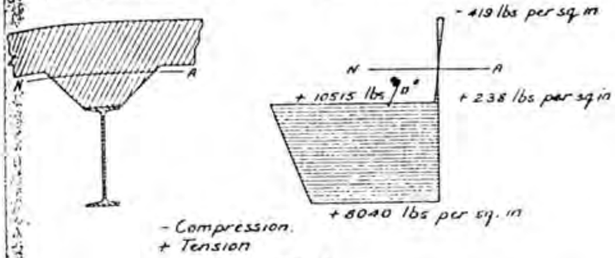


Fig. 5

tion after loading and unloading was evident. These factors, although present, were not considered excessive for the type of structure under test; based on distributed load deflection formula the amount of set after several loadings corresponded to a stress of about 330 lb. per sq. in. in the steel alone, which indicates the minor importance of this factor. Browne and Sharpe dials were used for measurement of deflection, supported by temporary staging under the bridge deck.

After making allowance for the distribution of the concentrated live load by the slab and the effect of the steel reinforcement on Young's modulus for the slab, the central deflection for the test load applied symetrically was calculated at 0.087 in. As previously stated the measured deflection was 0.059 in.; this serves to show the extraordinary stiffness of the section. The economy in steel and the absence of unsightly deflections are features which make this type of construction a proposition worthy of consideration for spans up to 60 or 70 ft. for double-track road bridges and for even greater spans in single-track bridges.

The Design and Construction of Composite Slab and Girder Bridges.

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*Junior.**

Summary.—The paper deals with the design of composite slab and girder bridges under the following headings :—
1. Introduction ; 2. Notation ; 3. Properties of the longitudinal member ; 4. Application to bridge design ; 5. Construction of composite beam bridges ; 6. Economics of the structure ; 7. Conclusion and 8. Acknowledgment.
An Appendix deals with the question of continuous beams on elastic supports.

1. Introduction.

In an attempt to develop economic methods for the design and construction of single and double track road bridges, close attention has been paid to this phase of the Public Works Department's activities in connection with the replacement of the wooden bridges throughout Tasmania by permanent structures.

The introduction of concrete slab decks for bridges has made a thorough investigation into the methods of design developed for timber decks and later applied to concrete decks essential. Despite the apparent simplicity of the problem, it becomes, in reality, extremely complicated owing to the complex nature of the distribution of a slab deck to elastic beams acting as supports.

In a paper† by G. D. Balsille, A.M.I.E.Aust., a description was given of the existing data and methods available for the design of simple bridges with concrete decks. In a second paper dealing with the distributing effect of the slab, read before the Tasmania Division of The Institution, an experimental method of obtaining the reactions to longitudinal supporting members for various ratios of stiffness of slab to beams was described.

Though closely allied to the distribution problems of the above papers, this paper is mainly a discussion of matters concerning the design and construction of a new type of bridge which consists of steel beams, in the direction of the span, carrying a concrete deck, which, with the steel, forms composite beams, the horizontal shearing forces in the section being provided for by the introduction of suitable steel reinforcement. The main features of the type of bridge to be described, as compared with other forms of road bridges, are :—

(1) The increased economy due to :
(*a*) the more effective use of the concrete of the deck slab ;

*This paper No. 470, originated in the Tasmania Division of The Institution and will be presented before the Engineering Conference in Hobart in February, 1934.

†See The Journal, Vol. 5, No. 2, February, 1933, p. 60.

(*b*) the uniform tensile stress throughout the steel; and
(*c*) the suitability of rolled, as against more costly fabricated sections, for relatively long spans.

(2) The absence of unsightly deflections.

2. Notation.

I	moment of inertia	(inches ^{4})
Z	modulus of resistance	(inches ^{3})
E_s	Young's modulus for steel	
E_c	Young's modulus for concrete	
n	ratio $\frac{E_s}{E_c}$	
f_{shear}	shear stress (lb. per sq. in.)	
f_s	steel stress (lb. per sq. in.)	
f_c	concrete stress (lb. per sq. in.)	
M	bending moment (inch tons)	
P	propping force (tons)	
S	vertical shear force (tons)	
s	horizontal shear force (tons)	
y	distance from neutral axis to any layer (inches)	
w_1	weight of beam, cover plate and stirrups (tons per ft. run)	
w_2	weight of formwork and deck slab (tons per ft. run)	
w_3	weight of formwork alone (tons per ft. run)	
[illegible]	deflection (inches)	

3. Properties of the Longitudinal Member.

In the type of bridge which consists of a concrete slab supported by steel joists, a type which is very economical owing to the simplicity of the formwork, the concrete slab itself acts as a beam in the direction of the span with the piers or abutments as supports. Experiments have shown that the adhesion between the concrete and the upper face of the flanges of the joists is practically negligible, thus it can be assumed that the joists and slab bend independently of one another. The modulus of the section as a whole is therefore the sum of the moduli of the steel and concrete sections. The introduction of suitable shear reinforcing at the surface of contact of the steel and concrete serves to transform the section composed of two independent parts as described above to a composite section of greatly increased strength. This is the basic principle involved in the design of the particular type of bridge with which this paper is concerned.

Thus the longitudinal member consists of a steel beam connected to a slab of concrete, and its form must be such as to allow it to act as the main supporting member and at the same time as the deck slab. As far as the steel section is concerned, a rolled steel joist is suitable both on the grounds of its suitability for supporting the formwork and of economy. Steel stirrups electrically welded to the top flange provide the shear reinforcing.

The concrete section must be symmetrical, the thickness of the concrete being proportioned from considerations of deck slab design. Small variations in the thickness of the deck slab have only a small effect on the concrete stress due to longitudinal bending owing to the fact that this variation in thickness alters the value of y_c (max.) for the section. If it is necessary to reduce the maximum concrete stress the width of the section of the slab attached to each beam must be increased. The introduction of a concrete haunch between the joist flange and the slab greatly increases the modulus of the composite section and more than offsets the increased cost of formwork. The fact that a symmetrical section is required necessitating cantilever supports to the formwork for the outside members is to some extent an advantage in that the distribution of dead load to the longitudinal members is approximately the same for each. It therefore obviates the necessity of special methods of construction necessary to ensure that the distribution of the dead load of the liquid mass of concrete is the same as for the concrete in the form of a stiff slab—the form for which the most economical design of the longitudinal members can be obtained.

A typical section is shown in Fig. 1. It consists of a 24 in. x $7\frac{1}{2}$ in. x 90 lb. rolled steel joist carrying a 6 in. x $\frac{3}{4}$ in. cover plate welded to the bottom flange, a $7\frac{1}{2}$ in. concrete haunch with side slopes of 45° and a 7 in. slab with a width of 93 in. per joist. The moment of inertia of the composite section is calculated by replacing the concrete by the equivalent area of steel (taking n, the ratio of Young's modulus of steel to concrete as 12).

For the composite section

Height of neutral axis	= 27.05 in.
Moment of inertia	= 15,470 in.^{4}
Section modulus for steel stress	= 572 in.^{3}

For steel alone

Height of neutral axis	= 10.95 in.
Moment of inertia	= 3032 in.^{4}
Z for top fibre	= 219.7
Z for bottom fibre	= 276.9

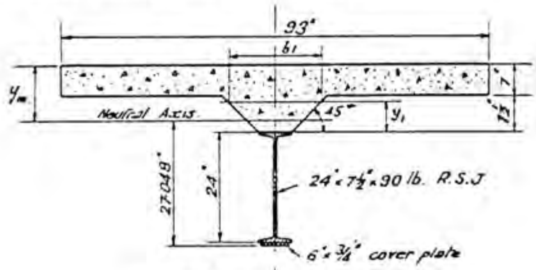

Fig. 1.—*Typical Section.*

The variation in the value of n for different classes of concrete has a considerable effect on the strength of the section and for any particular design it is desirable to make a close estimate of its expected value. For lower values of n there is a relative increase in the strength of the concrete to the strength of the steel, the neutral axis of the section moves away from the slab, tending to increase the maximum concrete stress. Since, however, the Z value of the section is increased and the stress is carried by a higher strength concrete the effect is not detrimental. Later in the paper a graph is given showing the variation in stresses for different values of n for a particular system of loading applied to a given section.

TABLE I.

The value of n is taken as 12.

Beam	Slab	Y_s	I	Z c.s.	Z r.s.j.	$\frac{Z_{c.s.}}{Z_{r.s.j.}}$
24in. x $7\frac{1}{2}$in. x 90lb. 2-9in. x 1in. C.P.	90in x 7in	27.28	21176	775	405	1.91
22in. x $7\frac{1}{2}$in. x 75lb.	,,	26.13	9076	348	153	2.27
20in. x $6\frac{1}{2}$in. x 65lb.	,,	24.55	7054	287	123	2.34
18in. x 6in. x 55lb.	,,	23.00	5291	230	94	2.45
16in. x 6in. x 50lb.	,,	21.53	4214	196	77	2.55
15in. x 6in. x 45lb.	,,	20.82	3779	182	66	2.76
14in. x $5\frac{1}{2}$in. x 40lb.	,,	19.87	3056	154	54	2.85

It is interesting to note the marked increase in strength of the composite section over the steel section alone as

illustrated by Table I. Unless special steel reinforcing is provided to carry the shear stresses no tee-beam action is developed so that in these cases the effect of the concrete is neglected. The longitudinal members consist of rolled steel joists carrying 7 in. x 90 in. concrete slabs on haunches.

Shear stresses in the composite section can be calculated in the usual manner by the equation,

$$f_{\text{shear}} = \frac{S}{I\,b_1}\int_{Y_1}^{Y_m} by\,\delta y \qquad \text{.................................(1)}$$

In view of what follows, it should be noted that when the section is propped, the shear must be calculated in two parts.

(1) due to reversed prop loads; and
(2) due to live loads.

By evaluating equation (1) for any particular section, the horizontal shear stress on any plane is obtained in terms of the vertical shear.

The maximum shear stress in the concrete occurs at the junction of the concrete and steel, owing to the minimum breadth occurring there and to the neutral axis being close to the top of the section.

The amount of steel reinforcing required in the concrete portion of the section is therefore determined by the shear on this plane and although the shear for planes above this is progressively less, it is not practicable to reduce this amount owing to the necessity of obtaining sufficient bond for the steel.

4. The Application to Bridge Design.

It will be obvious that, if the formwork for the concrete is carried from the steel joists, when the concrete is placed dead loads will be carried by the steel alone. In order to make the composite section effective for dead as well as for live loads it is necessary to support the steel joists in some way until the concrete has set. A further advantage can be gained by the use of temporary supports in that a negative bending moment can be applied to the steel by propping it in a suitable manner to give an initial camber.

After the concrete has set and the props are removed the resulting stresses in the steel are tension throughout, and by careful adjustment of the amount of initial upward deflection it is possible to make the steel stresses under maximum load conditions practically uniform throughout, thus utilizing the steel to the best possible advantage.

Additional shear stresses are thereby introduced, but the saving in steel in the joist more than offsets the additional shear steel required.

The first step in the design of a longitudinal member for a bridge is the determination of the bending moments and shear forces acting on it. This involves a knowledge of the dead and live loads acting on the structure; in this respect this type of bridge is no different to any other form of concrete slab and girder bridge, in that an accurate determination of the distribution presents a problem of very great difficulty.

Actually the structure consists of an elastic slab on elastic supports, the elasticity of the slab being approximately constant throughout the span length, and the elasticity of the supports varying from zero at the piers to a maximum at the centre of the span. Under such varying conditions the load distribution is very different for different parts of the span and although at first sight it might not appear necessary to go to such refinements in design as to take account of these differences, yet in so far as the design of the deck slab is concerned a close investigation of the problem shows it to be very necessary in that even a slight departure from the condition of rigid supports might cause a total reversal of bending moment in the slab.

One of the difficulties of the general problem of load distribution by slab decks supported by beams is the lack of consistent information regarding the stiffness of reinforced slabs. If accurate information of this nature were available, or if it could be determined by analytical methods, there would be little difficulty in arriving at a reasonably accurate design. Because of this lack of information, it is desirable to resort to models of a proposed structure, but unless such models are almost exact replicas of the actual structure the introduction of new variables complicates the results to such an extent that they can only be used as a general guide, and are unsatisfactory from the point of view of design. The reason why there is very little useful information on the subject is to be found in this wide divergence of test pieces from the conditions obtained in the actual structure.

The distribution of the loads to each of the main longitudinal members varies, of course, with their number. The number of beams required depends on the width and to a lesser extent on the span of the bridge. It is found that a spacing of between 6 and 9 feet is convenient so that for ordinary widths of road bridge 2, 3, 4 or 5 longitudinal members are required. Since it is advantageous both from the points of view of design and construction to have similar members, it is economical to arrange them so that the maximum bending moment to each is, as far as possible, the same.

On account of its close connections to the problem in hand, extensive reference will be found, in what follows, to the reactions of a continuous beam on elastic supports. The appendix gives an outline of the general method involved in obtaining the reactions and support bending moments for an elastic beam on any number of supports and also the solution for the 3, 4 and 5 beam types. The 2 beam type is statically determinate in this connection although this is not the case when the beam is replaced by a slab.

In order to illustrate the method of design, the four beam bridge shown in section in Fig. 2 will be examined. The span is 60 feet; the live load consisting of a crusher train of $34\frac{1}{2}$ tons total weight and a passing 10 ton truck. is also shown in the figure. A live load of 80 lb. per sq. ft. is allowed on the foot paths. The 7 in. concrete deck slab carries a 20 ft. roadway and two 4 ft. footpaths. The longitudinal members are spaced at 7 ft. 9 in. centres and

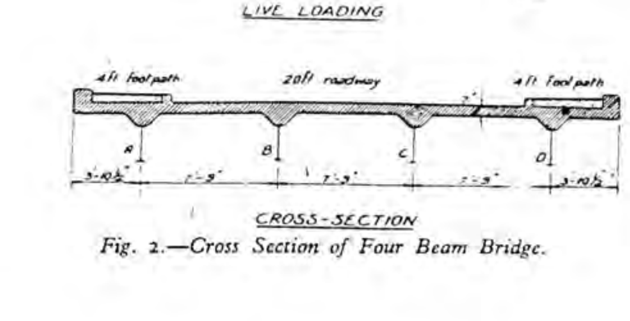

Fig. 2.—Cross Section of Four Beam Bridge.

are similar to the sections for which the properties are calculated earlier in the paper. The stresses in the outside members and the moments in the deck slab will be calculated.

As a preliminary step in the design of this bridge, influence lines for reactions are obtained analytically for a beam on four elastic supports for various values of the ratio $x = \frac{\text{stiffness of slab}}{\text{stiffness of supports}}$ the stiffness of the slab being defined as the central concentrated load required to give unit deflections over a span of $2l$, where l = spacing of the longitudinal members, and the stiffness of the support as the load required to give unit deflection of the longitudinal member over the span length, i.e., 60 feet. From various experimental information available, the values of x for the structure can be estimated. By using the corresponding reaction influence lines, obtainable from the equations of the appendix, the maximum reactions to the longitudinal members can be obtained for the dead and live load systems. It will be noticed that an approximation is involved here in that although the analytical solution for reaction influence lines is for a beam on elastic supports, these equations are actually applied to a slab on elastic supports. It is necessary to make this approximation since the analytical solution for the slab on elastic supports is not available. The results, however, are sufficiently accurate for a preliminary design of the cross section of the longitudinal member to carry the loads at satisfactory working stresses. In order to check the validity of the methods used in determining the load distribution to the beams, a $\frac{1}{8}$ scale model of this bridge has been built and extensive deflection experiments made on it. It consists of 4 in. electrically welded plate girders with $1\frac{1}{4}$ in. x $\frac{3}{16}$ in. flanges and $3\frac{5}{8}$ in. x $\frac{1}{8}$ in. webs. The lower flange carries a 1 in. x $\frac{1}{8}$ in. cover plate. A deck slab $1\frac{3}{8}$ in. thick with 10 and 12 gauge wire as deck reinforcing is cast on the beams. The concrete haunches are $1\frac{1}{4}$ in. high with 45° side slopes.

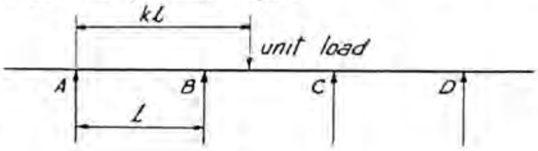

Fig. 3.

TABLE II.

Position of Load.			Reacts.ion	
k	A	B	C	D
— 0.25	0.752	0.291	0.048	— 0.095
0.0	0.683	0.309	0.078	— 0.070
0.5	0.477	0.369	0.161	— 0.007
1.0	0.326	0.370	0.232	0.072
1.5	0.180	0.320	0.320	0.180
2.0	0.072	0.232	0.370	0.326
2.5	—0.007	0.161	0.369	0.477
3.0	—0.070	0.078	0.309	0.683
3.25	—0.095	0.048	0.291	0.752

Note.—Position of load is expressed in terms of kl measured from support A. See Fig. 3.

Table II. gives the reactions for the centre span section of the model, obtained by measuring deflections under each of the four longitudinal members as a single concentrated load is moved in steps across the deck slab. Individual deflection readings are expressed as a proportion of the sum of the four readings, the result being taken as a measure of the reaction from the slab to the particular beam for which the deflection is measured. This is not necessarily correct as the load distribution probably varies for the different beams. The results obtained, however, appear to indicate that in this case the assumption is allowable.

The influence lines for reactions obtained in this way for the centre line section of the span are shown in Fig. 4. Plotted on the same diagram for a value of $x = 9$ are the reaction influence lines for a beam on elastic supports. The figure serves to illustrate the distributing effect of the slab as compared with the beam.

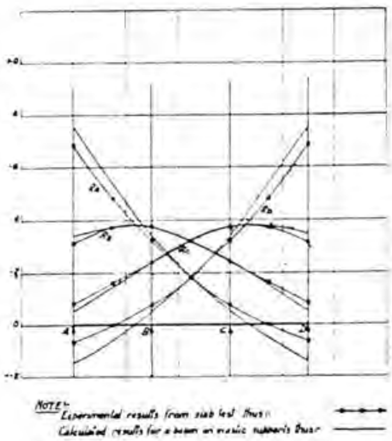

Fig. 4.—*Influence Lines for Reactions,* $\frac{1}{2}$ *Span.*

Although this experimental method of determining reactions would prove satisfactory for a beam on elastic supports it is not a true indication of the distribution of the load by a slab, as no account is taken of torsional moments which are negligible for the beam but considerable for the slab. The method of adjusting the results to allow for this torsion is illustrated by the following example. Test results from the model give the following reactions for a unit load over the left hand beam on the centre line section of the span.

$$R_A = 0.683 \quad R_B = 0.309 \quad R_C = 0.078 \quad R_D = -0.070$$

The forces acting on the system are shown in Fig. 5.

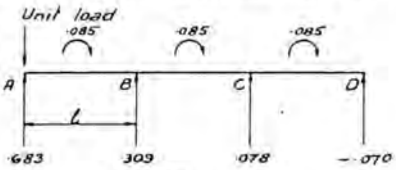

Fig. 5.

Any such system of forces must be in equilibrium. Resolving the forces in a vertical direction:—

$$0.683 + 0.309 + 0.078 - 0.070 = 0.$$

Taking moments about A

$$0.309 \times 1 + 0.078 \times 2 - 0.070 \times 3 = M_T$$
$$0.309 + 0.156 - 0.210 = M_T$$
$$0.225 = M_T \qquad \text{taking } l = \text{unity.}$$

M_T is the moment which must be introduced to the system to give equilibrium. Actually it is supplied by the torsional resistance of the vertical faces of any transverse strip of

the slab and therefore not shown as a reaction by the deflection measurements. This torsional moment in a slab under load constitutes one of the fundamental differences between a slab and a beam. If the above reactions are adjusted in terms of this moment they will correspond to the reactions obtained for a beam under the same condition of loading.

Assuming M_T evenly distributed over the sides of a transverse strip of the slab, the moment for a length l of the strip $= \frac{0.255}{3} = 0.085$ acting in the direction shown in Fig. 5. Each of these component moments can be resolved into a pair of equal and opposite forces of 0.085 acting at the adjacent supports. Summing the forces at each support the adjusted reactions are obtained :—

$$R_A = 0.683 + 0.085 = 0.768$$
$$R_B = 0.309 - 0.085 + 0.085 = 0.309$$
$$R_C = 0.078 - 0.085 + 0.085 = 0.078$$
$$R_D = -0.070 - 0.085 = -0.155$$

Adjusting the corresponding reactions, in this way, for each position of the load, a new set is obtained; this set is shown plotted in Fig. 6. Also plotted on the same diagram are the reactions as calculated from the analytical equations for the four beam bridge for a value of $x = 9$. (This value of x is obtained by substituting values of reactions taken from the adjusted test results in the appropriate equations and solving for x).

What discrepancy there is between the two sets of curves may be partly due to the assumption that the torsional moment is evenly distributed.

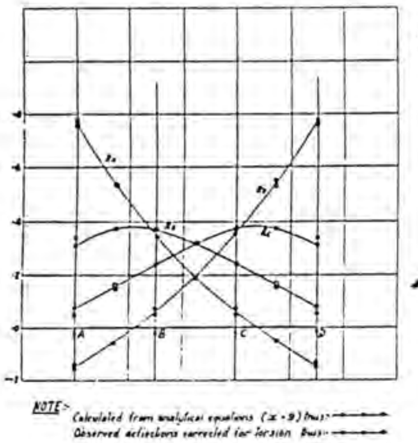

Fig. 6.—Reaction Influence Lines.

Reaction influence line curves obtained experimentally for the sections at $\frac{1}{8}$, $\frac{1}{4}$ and $\frac{3}{8}$ of the span length confirm the supposition that the stiffness of the slab is approximately constant throughout the whole span whereas the stiffness of the longitudinal member varies from zero at the abutments to a maximum at the centre of the span. It is a simple matter to calculate the stiffness of the longitudinal member at any particular point; the value of x at the centre is 9. It therefore follows that x varies approximately as the reciprocal of the stiffness of the longitudinal member, i.e., is 0, 0.494, 1.723, 5.062 and 9 at the abutment, $\frac{1}{16}$, $\frac{1}{8}$, $\frac{1}{4}$ and $\frac{1}{2}$ span points respectively.

Actually the values obtained experimentally were 0, 1.5, 3, 5.7 and 9 and reaction influence lines for the above values of x are plotted in Figs. 7 to 10. They serve to show the high distributing effect of the slab over the greater portion of the span, also the sudden change in distribution for a slight departure from the condition of rigid supports and the more gradual change in distribution when the value of x becomes appreciable. These changes in distribution are of extreme importance as far as the design of the deck slab itself is concerned.

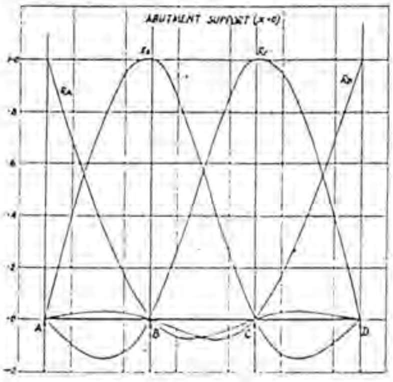

Fig. 7.—Reaction Influence Lines.

The live load bending moments in the slab can now be calculated. The maximum moment curves obtained by drawing envelopes over the series of bending moment diagrams obtained by moving two unit loads spaced at 6 feet centres from the outside beam to the centre of the deck in steps can be obtained for the sections of the slab for which the values of x are noted above. The bending moment for any point, in terms of wl where w is a wheel load, is simply obtained by taking moments of the external forces as obtained from the corrected reaction diagram. (If the value of x is known, the reaction diagram can be determined analytically.)

Fig. 11 shows a diagram of the maximum live load moments for the inside and outside slab spans and the inside supports plotted against the span length. A positive moment is defined as one producing compression in the top of the slab. The coefficient of maximum moment in the inside spans increases from 0.149 at the abutment support to 0.566 at $\frac{1}{2}$ span and in the outside span from 0.166 to 0.476. The support moment coefficient varies from —0.174 at the support to —0.150 at $\frac{1}{2}$ span. An interesting feature of this diagram is the curve showing the ratio of maximum negative to maximum positive moment in the slab. This ratio is practically constant at 0.28 over $\frac{7}{8}$ of the span length; the ratio of steel area required in the top and bottom of the slab for live load moment is, of course, the same. The moments shown in the graph must be distributed over the effective width of the slab and added to the dead load moments (which are constant for any section of the span) to obtain the moments for design purposes.

The introduction of the idea of effective width is necessary in order to form a practical basis for the design of the reinforcement. Extensive tests (see Vol. 7 No. 1 *Public Roads*) indicate that the effective width of a slab can be expressed in terms of the slab width, which in this case is the distance between the longitudinal members. The results of these tests are applicable at the abutment where under a distributed load the slab has a point of contraflexure at each support but for other points in the

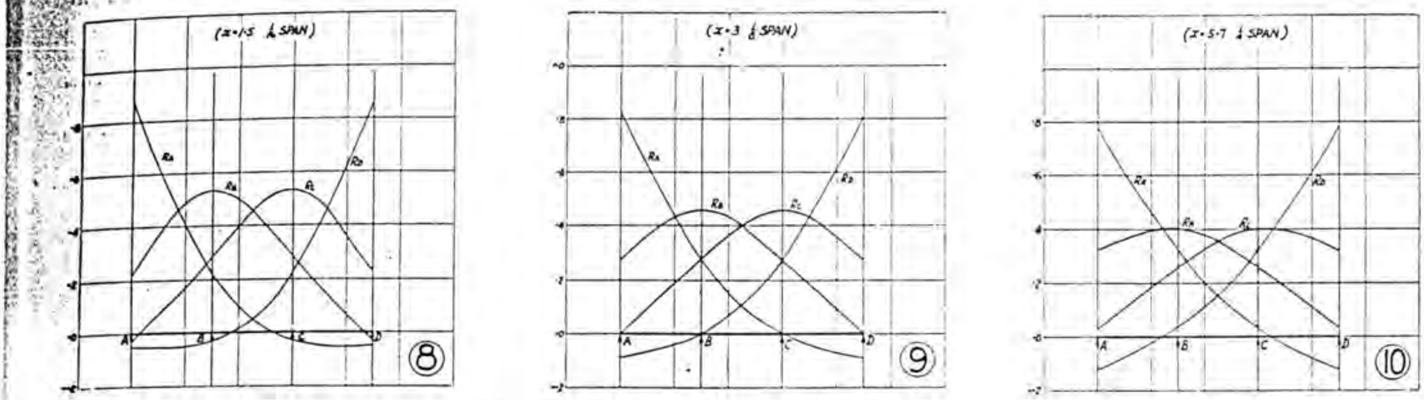

Fig. 8, *Fig* 9, *Fig.* 10.—*Reaction Influence Lines.*

span where, owing to the elasticity of the supporting members, the slab tends to bend in a smooth curve throughout its full width, the slab width mentioned above is increased with a consequent increase in the effective width. By comparison of the bending moment diagrams for a unit load on the slab at the abutment and at the centre of the span it is found that the effective width at the centre is approximately twice that at the abutments.

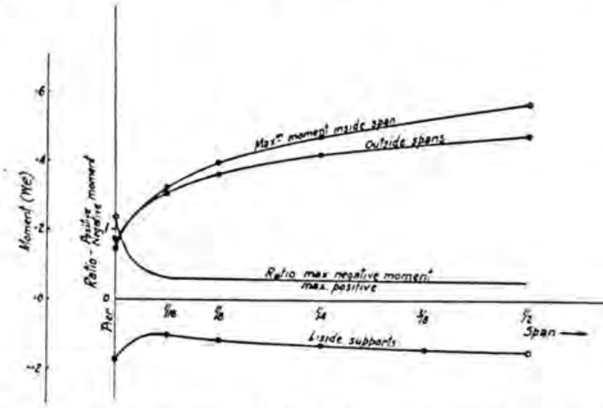

Fig. 11.—*Maximum Moments in Slab, Live Load Coefficients of Wl.*

The above analysis demonstrates the possible errors involved in assuming the longitudinal members of a bridge deck rigid. So much for the deck slab.

By using Fig. 4, the dead and live load reactions to the longitudinal members can be obtained by ordinary influence line methods. It is not necessary to consider the variation of the distribution near the abutment supports as over the greater portion of the slab (the portion where the loads produce the greatest moments) this variation is only slight. Having obtained the maximum wheel load reactions on any member by the above method the bending moment and shear forces acting on the member can be calculated and hence the stresses in the member due to these moments and shears.

To illustrate the method of calculation of the stresses, it is proposed to work out for the section already referred to, the dead load stresses due to the weight of the structure and the propping system and the live load stresses for the loading adopted as a standard for bridge design in the Public Works Department and shown in Fig. 2. The points which need consideration in deciding what form of propping system should be adopted are worth noting.

The propping forces required, even for long spans, are relatively small so that it is not a matter of dividing the load among a number of props. The important consideration is the stress distribution throughout the section in the direction of the span. It is possible to express analytically the stresses in terms of the live loads and the propping forces; the latter can then be evaluated for any particular distribution and applied by a suitable adjustment of the prop levels. In the example the system adopted consists of producing the initial stresses in the steel joist by the application of a single propping force at the centre of the span. Two additional props are then placed in contact with the cambered beam at the quarter points and maintained at this level while the formwork is fixed in position and the concrete placed. It will be seen that the stresses produced are not a maximum at the centre but at two points, one on either side of the centre; it is necessary to adjust the propping forces to suit the properties of the section so that a reasonably good distribution is obtained.

The value of the initial stresses to be placed in the steel joist are fixed arbitrarily but although any initial value can be obtained by adjustment of the prop levels, this value will be modified by the loads due to first the formwork and later the concrete of the deck. Suppose the upward deflection is required to produce a compressive stress of approximately 3 tons per sq. in. in the steel. If this stress is produced by a single prop at the centre of the span:

$$\text{then } \delta = \frac{Pl^3}{48E_s I}, \text{ where } P = \text{propping force} \ldots\ldots (2)$$

$$\text{and } f = \frac{My}{I} = \frac{Ply}{4I}$$

$$\therefore \delta = \frac{fl^2}{12Ey} = \frac{3 \times 2240\,(60 \times 12)^2}{12 \times 30 \times 10^6 \times 10.95} = 0.884\text{in.}$$

The downward deflection due to the dead load of steel joist, stirrups and cover plate is given by

$$\delta = \frac{5}{384}\frac{wl^4}{EI} = \frac{5 \times 0.046 \times 60\,(60 \times 12)^3 \times 2240}{384 \times 30 \times 10^6 \times 3032}$$
$$= 0.329\text{in.}$$

and the corresponding stress is 0.893 tons per sq. in., tension. If an upward camber equal the sum of these deflections i.e. $0.884 + 0.329 = 1.213$ in. is introduced by the central prop

an initial stress of $3 + 0.893 \times \frac{5}{4} - 0.893 = 3.224$ tons per sq. in. is obtained. The force on the central prop required to produce this deflection is given by

$$P = \frac{48E_sI\delta}{l^3} \qquad \text{(by transposing equation (2))}$$

$$= \frac{48 \times 30 \times 10^6 \times 3032 \times 1.213}{(60 \times 12)^3 \times 2240} = 6.334 \text{ tons}$$

Two additional props are now introduced at the quarter points and placed in contact with the lower flange of the joist. The support bending moments and reactions for the system of loading shown in Fig. 12 are as follows:

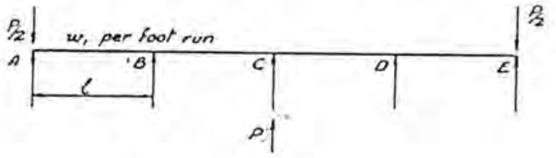

Fig. 12.

where w_1 = weight of beam, cover plate and stirrups per foot = 0.046 tons

$$M_A = M_E = 0$$

$$M_B = M_D = \frac{P}{2}.l - \frac{w_1l^2}{2} = \frac{6.334}{2} \times 15 - \frac{0.046 \times 15^2}{2} = 42.330 \text{ ft. tons.}$$

$$M_C = \frac{P}{2}.2l - 2w_1l^2 = \frac{6.334}{2} \times 30 - 2 \times 0.046 \times 15^2 = 74.310 \text{ ft. tons.}$$

$$R_A = R_E = \frac{P}{2} + \frac{4w_1l}{2} = \frac{6.334}{2} + \frac{4 \times 0.046 \times 15}{2} = -1.787 \text{ tons.}$$

$$R_C = P = 6.334 \text{ tons.}$$

The shear and bending moment diagrams for this system are shown in Fig. 13.

The three props are maintained at their original level while the formwork and deck reinforcing is placed in position and the deck slab cast. For this condition the joist acts as a continuous beam of four equal spans, of length l feet, carrying a distributed load of w_2 tons per foot run.

The support bending moments and reactions are

$$M_A = M_E = 0$$

$$M_B = M_D = 3/28w_2l^2 = \frac{3}{28} \times 0.500 \times 15^2 = 12.054 \text{ ft. tons}$$

$$M_C = \frac{1}{14}w_2l^2 = \frac{1}{14} \times 0.500 \times 15^2 = 8.036 \text{ ft. tons}$$

$$R_A = R_E = \frac{11}{28}w_2l = \frac{11}{28} \times 0.500 \times 15 = 2.946 \text{ tons}$$

$$R_B = R_D = \frac{8}{7}w_2l = \frac{8}{7} \times 0.500 \times 15 = 8.571 \text{ tons}$$

$$R_C = \frac{13}{14}w_2l = \frac{13}{14} \times 0.500 \times 15 = 6.964 \text{ tons}$$

where $w_2 = 0.469 + 0.031 = 0.500$ tons per ft. run
$l = 15$ feet.

The bending moment and shear diagrams for this system are shown in Fig. 14.

Superimposing these bending moments and shearing forces on those of the previous system, the following total moments and reactions are obtained on the steel before removal of the props or formwork.

$$M_A = M_E = 0$$
$$M_B = M_D = 42.330 + 12.054 = 54.384 \text{ ft. tons}$$
$$M_C = 74.310 + 8.036 = 82.346 \text{ ft. tons}$$
$$R_A = R_E = -1.787 + 2.946 = 1.159 \text{ tons}$$
$$R_B = R_D = 8.571 \text{ tons}$$
$$R_C = 6.334 + 6.964 = 13.298 \text{ tons}$$

The total moment acting on the steel is shown in Fig. 15 (obtained by summing the diagrams of Figs. 13 and 14).

The maximum steel stresses at the centre of the span before removal of the props are therefore given by

$$f_{s1} = \frac{My}{I_s} = \frac{82.346 \times 13.80 \times 12}{3032} = 4.498 \text{ tons per sq. in.}$$

$$f_{s2} = \frac{82.346 (-10.95) \times 12}{3032} = -3.569 \text{ tons per sq. in.}$$

The corresponding stress diagram is shown in Fig. 16*a*.

When the concrete has set, the formwork and the props at the points B, C and D can be removed. The stresses resulting from the removal of the above falsework will be carried by the composite section. Suppose the formwork weighs, w_3 tons per foot run and that it is removed before

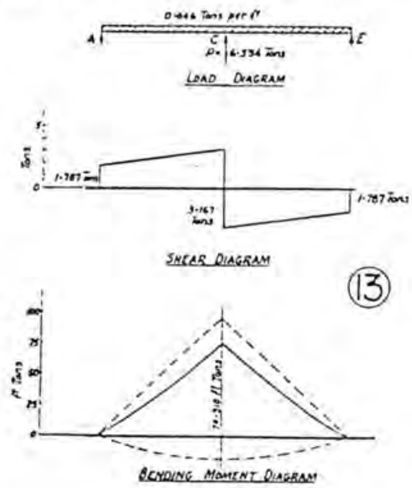

Fig. 13.

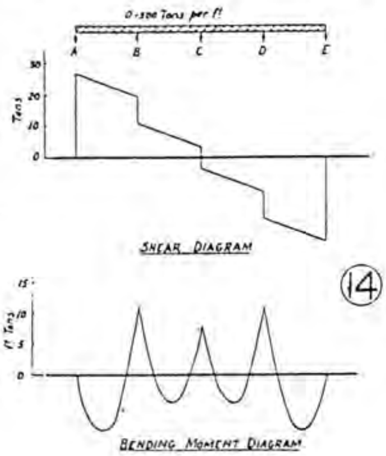

Fig. 14.

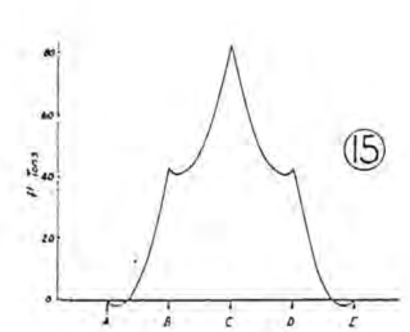

Fig. 15.—*Total Bending Moment Diagram. Props in Position and Concrete Placed.*

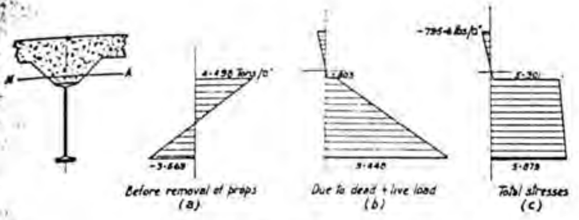

Fig. 16.

the props. Then moments and reactions at the supports due to its removal are:

$M_A = M_E = 0$

$M_B = M_D = -\frac{3}{28}w_3l^2 = \frac{-3}{28} \times 0.031 \times 15^2 = -0.747$ ft. tons

$M_C = -\frac{1}{14}w_2l^2 = -\frac{1}{14} \times 0.031 \times 15^2 = -0.498$ ft. tons

$R_A = R_E = -\frac{11}{28}w_3l = -\frac{11}{28} \times 0.031 \times 15 = -0.182$ tons

$R_B = R_D = -\frac{8}{7}w_3l = -\frac{8}{7} \times 0.031 \times 15 = -0.531$ tons

$R_C = -\frac{13}{14}w_3l = -\frac{13}{14} \times 0.031 \times 15 = -0.432$ tons

Thus the prop loads removed are

$R_B = R_D = 8.570 - 0.531 = 8.039$ tons

$R_C = 13.298 - 0.432 = 12.866$ tons

They are shown diagramatically in Fig. 17 together with the reactions at A and E caused by their removal.

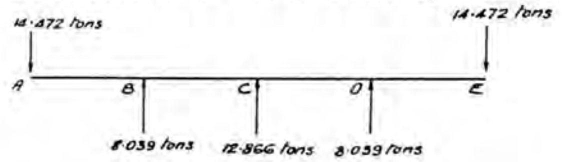

Fig. 17.

The reversed bending moments (obtained by taking moments of the forces of Fig. 17) are therefore

$M_A = M_E = 0$

$M_B = M_D = -14.472 \times 15 = -217.080$ ft. tons

$M_C = -14.472 \times 30 + 8.039 \times 15 = -313.575$ ft. tons

They are shown plotted in Fig. 18.

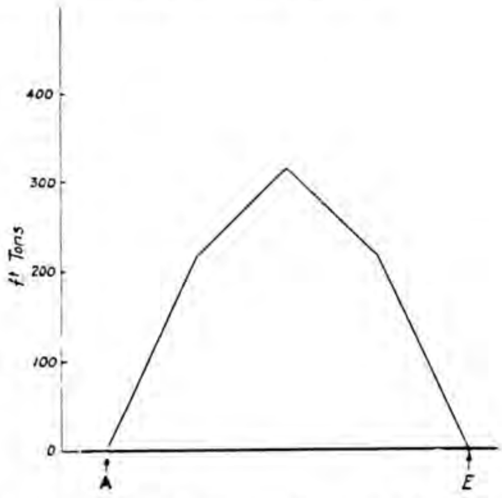

Fig. 18.—*Reversed Bending Moments Carried by Composite Beam when Props are Removed.*

It remains to calculate the live load bending moment before the total stresses in the section can be calculated. The live loads are placed in the centre of their respective traffic lanes and the wheel reaction ordinates read from the influence line diagram of Fig. 4. Multiplying the ordinates by the moments produced by the corresponding train of wheel loads on a 60 ft. span, i.e., 150 ft.-tons for the crusher train and 62.5 ft.-tons for the truck, the maximum moment to each longitudinal member is obtained. This moment is increased by 15% for impact but as no allowance is made for the distributing effect in the direction of the span of the slab on the concentrated loads, the impact allowance is actually in the vicinity of 25% to 30%. It is assumed that the live load on the foot path is negligible when the full vehicular live load is on the bridge.

Thus for the outside beams at the centre section of the span:

Live load moment =

$-1.15 \{ 150 (0.493 + 0.243) + 62.5 (0.131 = 0.004) \}$

$= -1.15 \{ 110.50 + 7.98 \}$

$= -136.252$ ft. tons.

Hence the total moment for this section is

$M_C = -313.575 - 0.498 - 136.252 = -450.325$ ft. tons.

and the stresses due to this moment acting on the composite section are:

$$f_c(\text{max.}) = \frac{M_c y}{nI} = \frac{-450.325 \times 12 \times 12.20 \times 2240}{15471 \times 12} = -795.6$$

lb. per sq. in.

$$f_c(\text{min.}) = \frac{-450.325 \times 12 (-2.30) \times 2240}{15471 \times 12} = 149.8$$

lb. per sq. in.

$$f_s(\text{min.}) = \frac{M_c y}{I} = \frac{-450.325 \times 12 (-2.30)}{15471} = 0.803 \text{ tons per sq. in.}$$

$$f_s(\text{max.}) = \frac{-450.325 \times 12 (-27.048)}{15471} = 9.448 \text{ tons per sq. in.}$$

These stresses are shown plotted in Fig. 16*b*. These stresses can be superimposed on the steel stresses before removal of the props, giving final stresses at the centre of the span of

$f_c(\text{max.}) = -795.6$ lb. per sq. in. (compression)

$f_c(\text{min.}) = 149.8$ lb. per sq. in. (tension)

f_s (top of steel section) $= 4.498 + 0.803 = 5.301$ tons per sq. in. (tension)

f_s (bottom of steel) $= -3.569 + 9.448 = 5.879$ tons per sq. in. (tension)

The corresponding stress diagram is plotted in Fig. 16*c*.

The central deflection under dead and live load conditions is obtainable from the steel stresses at this point thus

$$\delta = \frac{f_s l^2}{12Ey} = \frac{(5.879 - 5.301)\frac{10.95}{24.75} \times (60 \times 12)^2 . 2240}{12 \times 30 \times 10^6 \times 10.95} = 0.075\text{in.}$$

(approximately for the actual conditions of loading)

The deflection is downwards because the stress is greater in the bottom flange than in the top. If an accurate determination of this deflection is required the deflection for each stage of the loading should be calculated and the results summed.

It is desirable to examine the steel stresses adjacent to the centre of the span under full load conditions. When propped and the concrete is placed $M_C = 82.346$ ft. tons and $M_B = M_D = 54.384$ ft. tons. There is a distributed

load of 0.546 tons per foot throughout. Hence the bending moment at x feet from C may be represented by the general equation.

$$M = \frac{15-x}{15} \times 82.346 + \frac{x}{15} \times 54.384 - \frac{0.546}{2}\, x\,(15-x)$$
$$= 82.346 - 5.959x + 0.273x^2$$

Multiplying by 12 to reduce the units to inch tons and dividing by the section modulus of the steel alone, i.e., $\frac{3032}{13.80} = 219.71$ at the top flange, and $\frac{3032}{-10.95} = -276.89$ at the bottom:

$$f_s \text{ (top flange)} = \frac{M}{Z} = 4.498 - 0.3254x + 0.01491x^2$$
$$f_s \text{ (bottom flange)} = -3.569 + 0.2582x - 0.01183x^2$$

The moment at x feet from the span centre line when props and formwork are removed is

$$M = -\left\{313.575 + 0.498 - 6.4164x + \frac{0.031}{2}.\ x\,(15-x)\right\}$$
$$= -\{314.073 - 6.1844x - 0.0155x^2\}$$

$$f_s \text{ (top flange)} = \frac{My}{I} = -\{314.073 - 6.1844x - 0.0155x^2\} \times$$
$$\times \frac{12\ (-2.298)}{15471} = 0.5598 - 0.01102x - 0.0000276x^2$$
$$f_s \text{ (bottom flange)} = -\{314.073 - 6.1844x - 0.0155x^2\} \times$$
$$\times \frac{12\,(-27.048)}{15471} = 6.5892 - 0.1297x - 0.000325x^2$$

Combining these with the previous stress equations the total dead load steel stresses are given by the equations

$$f_s \text{ (top flange)} = 5.0581 - 0.3364x + 0.01494x^2$$
$$f_s \text{ (bottom flange)} = 3.0202 + 0.1284x - 0.01216x^2$$

Assuming the maximum moment curve for live loads is a parabola, the moment at any point x ft. from the span centre line is $136.252\ \left(1 - \frac{x^2}{30^2}\right)$

and the corresponding stress equations are fs (top flange)

$$= -\left\{136.252 - 0.15139x^2\right\}\ \frac{12\,(-2.298)}{15471}$$
$$= 0.2429 - 0.000270x^2$$
$$f_s \text{ (bottom flange)} = -\left\{136.252 - 0.15139x^2\right\}\ \frac{12\,(-27.048)}{15471}$$
$$= 2.8586 - 0.003176x^2$$

Combining with the dead load stress equations we obtain the final stress equations.

$$f_s \text{ (top flange)} = 5.3010 - 0.3364x + 0.014669x^2$$
$$f_s \text{ (bottom flange)} = 5.8788 - 0.1284x - 0.01533x^2$$

Fig. 19 shows the live and dead load steel stresses for points adjacent to the centre of the span.

Design of Shear Reinforcing.

Since there is no shear in the composite section before removal of the formwork and props, the shear when they are removed is that due to these loads reversed, and is shown plotted from the load diagram in Fig. 20*a*. The shear due to the live load must be added to the above. Assuming each longitudinal member to carry the same proportion of shear force due to the live load as of bending moment, i.e., $\frac{127.2}{300} = 0.424$ for the crusher train and $\frac{9.32}{125} = 0.074$ for the passing truck; then maximum live load shear at end of span:

$$= \left(11 + \frac{49}{60} \times 6.75 + \frac{40}{60} \times 4.75 + \frac{26}{60} \times 2 + \frac{12}{60} \times 2 + \frac{4}{60} \times 1.25\right)$$
$$0.424 + \left(7 + 3 \times \frac{46}{60}\right) 0.074$$
$$= (21.030 \times 0.424) + (9.3 \times 0.074) = 9.605 \text{ tons}$$

and with the heaviest load at the centre the shear at this point

$$= \left(5.5 \times \frac{18}{60} + 11 \times \frac{30}{60} + 6.75 \times \frac{19}{60} + 4.75 \times \frac{10}{60}\right) 0.424$$
$$+ \left(7 \times \frac{30}{60} + 3 \times \frac{16}{60}\right) 0.074$$
$$= (10.079 \times 0.424) + (4.300 \times 0.074) = 4.592 \text{ tons.}$$

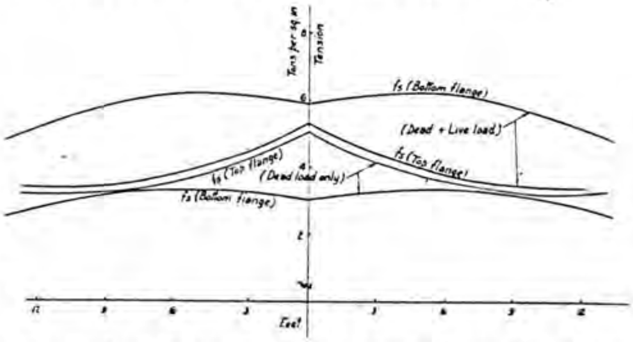

Fig. 19.—Steel Stresses for Points Adjacent to Centre Line of Span.

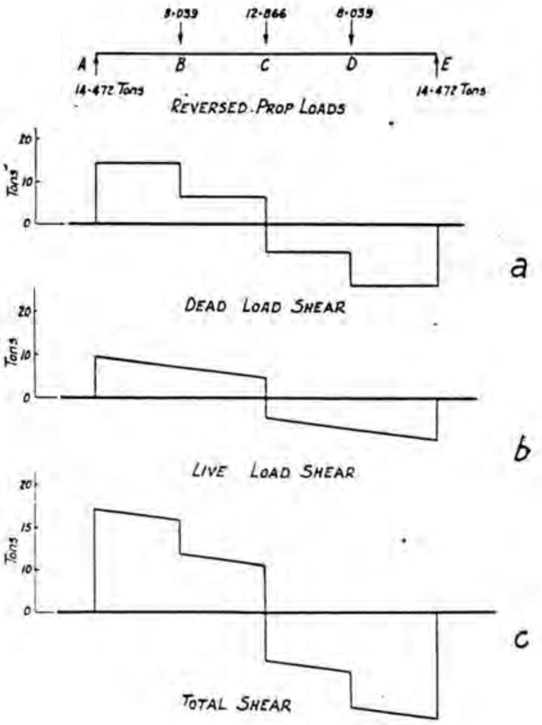

Fig. 20.

The live load shear can be assumed linear between these points. The corresponding shear diagram is shown in Fig. 20*b* and the total shear diagram in Fig. 20*c*.

The horizontal shear in the composite beam section is given in terms of the vertical shear of Fig. 20*a* by formulae (1) i.e.:

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$$s = f_{\text{shear}_1} = \frac{S}{I}\int_{y_1}^{y_m} by\delta y$$

It is necessary to reinforce the concrete in the section for the horizontal shear. The shear force s is a maximum at the plane of contact of the top flange and the concrete haunch; evaluating the equation for this plane

$$s = \frac{S}{15471}\left\{\frac{93}{12}\int_{5.20}^{12.20} y\delta y + \left(1.008 + 0.1667y\right)_{-2.30}^{5.20}\int_{-2.30}^{5.20} y\delta y\right\} = \frac{S}{15471}\left\{472.08 + 18.52\right\} =$$
$$= 0.0317S$$

Ordinary round steel reinforcing rods welded in the form of stirrups to the top flange of the joist, form a convenient type of reinforcement for this purpose, either the size or the spacing being varied to provide the required amount. The rods are bent up into the top of the slab and provided with hooks in order to develop the necessary bond. As to whether the rods should be designed for shear or tension stress depends on the angle at which they are inclined to the joist. The practice in the Public Works Department is to bend the rods up at an angle of 45° in two directions and design them for a tension stress on the cross sectional area of the rod.

Sufficient reinforcement must be provided to carry the shear in the concrete at the vertical sections between the flange and the stem of the tee beam formed by the composite section. It will be found that the deck reinforcing is sufficient for this purpose without the introduction of any extra material.

Since the process of welding the stirrups on one flange only tends to distort the joist it is convenient, in cases where the steel section requires strengthening by cover plates, to place the plate or plates on the lower flange only. Not only is this the most effective position to place the extra steel but it tends to correct the distortion effect noted above. There is some difficulty in welding round steel to a flat surface such as the flange of a joist; for this reason square reinforcing steel may be preferred, in which case this difficulty does not arise.

5. Construction of Composite Beam Bridges.

The general procedure to be followed in constructing the bridge superstructure has been outlined in the section dealing with the design. Having decided on the points at which the longitudinal members are to be propped and having calculated the propping forces required at these points, it remains actually to provide these forces. Since the propping force is fixed by an accurate adjustment of the level of a prop it is important that the prop should be of such a form that its original level once fixed is maintained during the progress of the work.

The method of supporting the props will vary with the natural conditions of the site but it can definitely be said that an arrangement whereby the prop level can be adjusted so as to maintain the correct propping force is essential. Screw jacks of suitable capacity supported either on piles or timber bents have proved satisfactory for this purpose, as any sinking of the prop due to the weight of the concrete deck slab can be adjusted as the work proceeds.

In order to measure the initial camber placed in the joist, and to ensure that this camber is maintained, a suitable means of measuring the level of the top flange of the joist must be adopted. It is desirable that such measurements should be made to $\frac{1}{100}$th of an inch; for this purpose a surveyor's level is of little use, particularly as the sight distance might be considerable. A simple method which has been used with success is to stretch a steel piano wire between adjacent piers an inch or so from the lower flange of each joist, clear of any falsework; if the wire is kept at the same tension by attaching a suitable weight to one end and passing it over a pulley attached to the pier, all deflections can be measured from this wire.

The necessity for wind and sway bracing is obviated when the composite beam type of construction is used, the concrete deck slab resisting these forces. Some form of bracing, however, is desirable during construction when the span length is greater than about 45 feet. The introduction of bracing which has a stiffening effect in the transverse direction leads to a complicated state of affairs as far as distribution is concerned, making it extremely difficult to compute the maximum moments and shearing forces carried by each longitudinal member. Disregarding the effect of the bracing on the main members there still remains the fact that the elasticity of these members under load is responsible for far higher stresses in the bracing than those for which it is usually designed.

The phenomena of plastic flow in concrete must be considered in designing this type of bridge. If this were appreciable it might be expected that the dead load compression in the concrete would be relieved at the expense of extra stress in the steel joist. Various experiments on plastic flow of concrete have given results which indicate that its effect is seriously to increase the stresses in reinforcing steel particularly in reinforced concrete columns, but as far as the 34 ft. span experimental bridge of this type is concerned the effects of plastic flow are small. Deflection measurements made over a period of 9 months gave the results shown in Fig. 21.

Any creep in the concrete would result in an increased deflection in the beams with a consequent increase in dead load steel stresses proportional to this deflection. Since the steel joist alone is comparatively slender an appreciable deflection is necessary to give any serious increase in stress in the steel; for the maximum sag measured in the above bridge the corresponding steel stress amounted to 1.046 tons per sq. in. The fact that at one stage the sag appeared to be decreasing is possibly due to a redistribution of stresses in the section due to the change in the modulus of elasticity of the concrete as its strength increases with age. On the other hand it may be due to temperature changes.

The use of pneumatic rammers for placing the concrete, enabling a reduction in the $\frac{\text{water}}{\text{cement}}$ ratio, should have the dual effect of increasing its ultimate strength and at the same time minimizing the plastic flow. Provided the stress in the steel joist due to plastic flow in the concrete can be estimated it is a simple matter to provide an equal and opposite stress in the steel by means of the props, thus neutralizing the effect. Results to date indicate that the effects of plastic flow in this class of structure are unimportant.

Plastic flow is probably closely connected with the value of Young's modulus for the material and in this con-

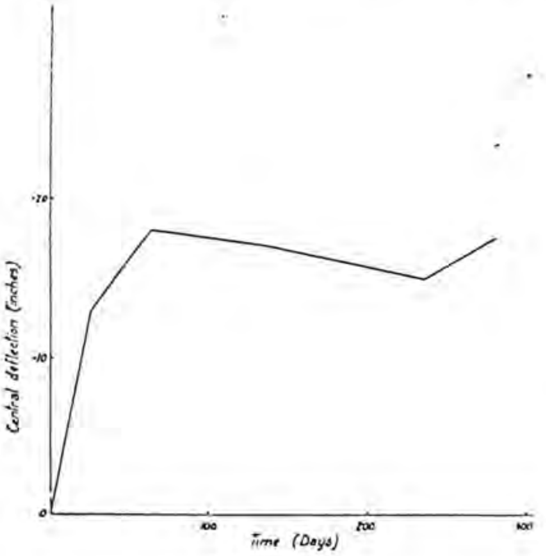

Fig. 21.

nection it is interesting to refer to Fig. 22 which shows the maximum steel and concrete working stresses (for the section treated throughout the paper) under full live and dead load, plotted against various values of n, the ratio of Young's modulus of steel/concrete. The factor of safety in the steel and concrete is also plotted against n. E for the concrete was taken to be proportional to its ultimate strength which in the light of at least some test data appears reasonable.

The steel stresses show a slight decrease for lower values of n, though this might not be the case with all sections examined, but there is an appreciable decrease in the concrete stresses and a marked increase in the factor of safety. A study of this graph shows the importance of a reasonably close determination of Young's modulus for the concrete used in the deck slab.

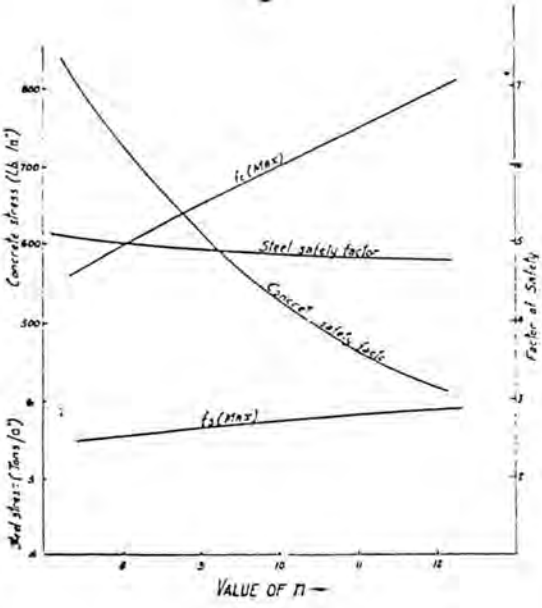

Fig. 22.—*Effect of n on Steel and Concrete Stresses.*

It might be noted that, in determining the type of wearing surface to be used on the deck, one formed by an increase in the thickness of the deck slab is a better proposition than a layer of some other material (other than a thin coating of a plastic material to reduce impact effect) since the extra concrete serves to increase the strength of the longitudinal member. The practice of placing a thick wearing surface on road bridge decks is to be condemned as it results in a saving of perhaps 1 inch in deck slab thickness at the expense of 3 inches of surface material with a consequent increase in dead load moment. This effect is of course more pronounced on long spans.

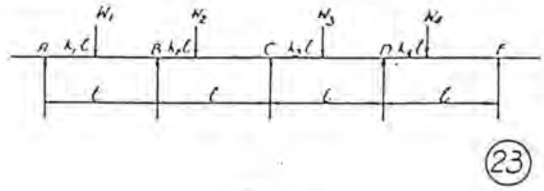

Fig. 23.

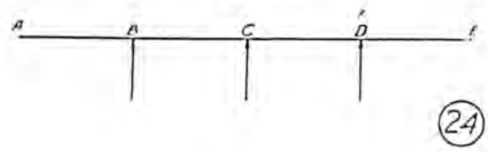

Fig. 24.

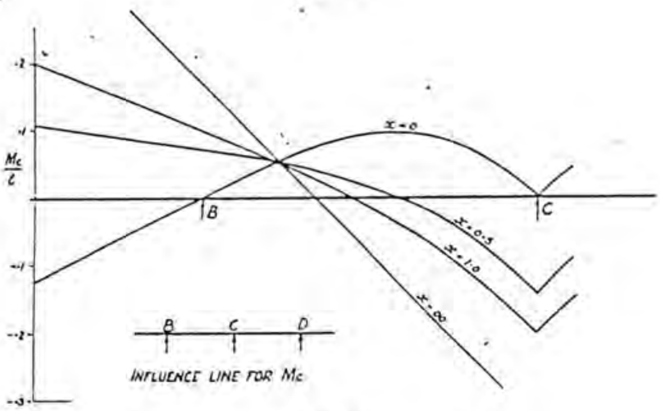

Fig. 25.

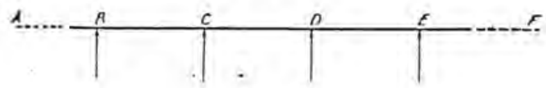

Fig. 26.

A feature of this type of bridge is its stiffness as measured by test compared with the calculated stiffness. The explanation is probably due to an underestimation of Young's modulus of the concrete and also to lack of knowledge regarding the distributing effect of the slab through which the test load is applied. Recent tests show that the error due to the latter reason would be quite sufficient to account for the discrepancy between the calculated and the actual values under test.

The rigid connection of the beams to the deck slab appears to increase the resistance of the bridge to impact effect, a noticeable feature of the structure being its elasticity under loads causing these effects.

6. Economics of the Structure.

A perusal of the section moduli of the composite sections of Table I. will indicate the economy in these sections as compared with the ordinary rolled sections. The only figures of cost available are those for the steel superstructure for the 34 feet single span experimental bridge which gave a saving of 29.5% over ordinary methods, and an estimated saving of 35% on the steelwork of the superstructure of a 427 ft. bridge of 7 spans.

7. Conclusion.

In conclusion, mention should be made of the question of slab bridge deck design which has been discussed in conjunction with the other matters of the paper. A review of the development of design methods for slabs for bridge decks shows a certain lack of appreciation of the problem in hand. Although it is an accepted principle never to place continuous structures on non-rigid supports it has frequently been done with bridge decks without any provision being made for the changes in bending moment which are bound to occur. A certain amount of research on the distributing effect of slabs supported by a number of beams has been undertaken, but little attention has been paid to the effect of this distribution on the slab itself with the result that the design of deck slabs along the lines accepted as standard practice involves at one and the same time a waste of reinforcing steel and a low factor of safety.

8. Acknowledgment.

The thanks of the author are due to G. D. Balsille, A.M.I.E.Aust., Director of Public Works, for his assistance and encouragement during the period in which the material for this paper was collected, and also to Professor A. Burn, A.M.I.E.Aust., of the University of Tasmania to whom the general solution for the beam on elastic supports given in the appendix is due, and without whose valued help this paper could not have been produced.

APPENDIX.

By Prof. A. Burn.

The Continuous Beam on Elastic Supports.

The usual equations for a continuous beam are derived on the assumption that the supports do not deflect. It may not be generally realized that when the supports are elastic, as for example when a beam is supported on a number of other beams, the reactions at the supports and the bending moments in the beam are very different to those occurring with rigid supports.

The following analysis gives a general method of treatment, and also the solutions for the special case of a uniform beam with a number of equal spans on 3, 4 and 5 supports.

The general method is based on the identity of the slope over a support for adjacent spans. The end slope in one span is expressed in terms of the load on the span, the end moments, and the end deflections, and equating values gives an equation involving the load on two spans, three moments and three deflections.

For elastic supports, the deflections are the reactions divided by the stiffness of the supports, the latter being defined as the load to produce unit deflection.

Each reaction in turn can be expressed in terms of the loads on two adjacent spans and the three moments at the end of those spans. On substitution, a five moment equation is obtained. The five moment equations for all interior supports together with the end moment conditions provide sufficient equations to enable the support moments to be determined.

Expression for End Slopes.—In the most general case where the beam is of variable cross section the use of the M/EI diagram gives the simplest expression for the slopes due to bending moments. Adopting the notation used by R. C. Robin in his paper on Statically Indeterminate Frames (The Journal, May, 1933) and writing the slopes for a span A B :—

$$i_A = -M_A A_1 \frac{l - x_1}{l} - M_B A_2 \frac{x_2}{l} + A_w \frac{l - x_w}{l} + \frac{y_B - y_A}{l}$$

$$i_B = M_A A_1 \frac{x_1}{l} + M_B A_2 \frac{l - x_2}{l} - A_w \frac{xw}{l} + \frac{y_B - y_A}{l}$$

The last terms are the additional slopes resulting from the deflections y_A and y_B of the supports.

Expression for Reactions.—If A, B, and C are three consecutive supports of spans l_1 and l_2, and r_B is the reaction which would occur at B if the spans were discontinuous, then

$$R_B = r_B + \frac{M_B - M_A}{l_1} + \frac{M_B - M_C}{l_2}$$

and if u_B is the stiffness of the support B

$$y_B = \frac{R_B}{u_B}$$

Case of Equal Spans and Uniform Beam.—Referring to Fig. 23, the load on each span is kl from the nearest support to the left, with the appropriate suffix.

For all spans, $x_1 = x_2 = l/_3$

$$A_1 = A_2 = \frac{l}{2EI}$$

$$x_w = \frac{(1 + k)\, l}{3} \qquad A_w = \frac{Wk\,(1 - k)\, l^2}{2EI}$$

For the second span :—

$$i_C = + \frac{M_B l}{6EI} + \frac{M_C l}{3EI} - \frac{W_2 k_2 (1 - k_1)(1 - k_2)\, l^2}{6EI} + \frac{y_c - y_B}{l}$$

For the third span :—

$$i_C = - \frac{M_C l}{3EI} - \frac{M_D l}{6EI} + \frac{W_3 k_3 (1 - k_3)(2 - k_3)\, l^2}{6EI} + \frac{y_D - y_C}{l}$$

Reactions.—

$$R_B = k_1 W_1 + (1 - k_2)\, W_2 + \frac{2\, M_B - M_A - M_C}{l}$$

$$R_C = k_2 W_2 + (1 - k_3)\, W_3 + \frac{2\, M_C - M_B - M_D}{l}$$

$$R_D = k_3 W_3 + (1 - k_4)\, W_4 + \frac{2\, M_D - M_C - M_E}{l}$$

Dividing these by u gives the deflections.

In order to simplify the final equation the symbols, $K_L = k (1 - k)(2 - k)$ and $K_R = k (1 - k)(1 + k)$ will be used. These quantities occur very frequently in problems on continuous beams and a table of values for intervals of 0.1 is therefore given.

k =	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
K_L =	0.171	0.288	0.357	0.384	0.375	0.336	0.273	0.192	0.099
K_R =	0.099	0.192	0.273	0.336	0.375	0.384	0.357	0.288	0.171

For distributed loads over a whole span the average values of k and K are used, namely $k = \frac{1}{2}$ and $K = \frac{1}{4}$ for interior spans.

The Stiffness of the Beam will be defined as $v = \frac{6EI}{l^3}$. This definition arose from consideration of the case of 3 supports and the load to give unit deflection at the centre with the centre support removed. It has proved the most convenient measure for other cases as well. Omitting intermediate steps, multiplying the slopes by $\frac{6EI}{l}$ equating and rearranging with $x = \frac{v}{u}$ leads to the equation :—

$$x\, M_A + (1 - 4x)\, M_B + (4 + 6x)\, M_C + (1 - 4x)\, M_D + x\, M_E =$$
$$K_{R_2} W_2 l + K_{L_3} W_3 l + xl \;\{k_1 W_1 + (1 - 3k_2)\, W_2 + (3k_3 - 2)\, W_3 +$$
$$+ (1 - k_4)\, W_4\}$$

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This is the five moment equation for the uniform beam on equal spans with equal stiffness of supports.

For plotting influence lines a unit load on one span only is taken, so that the right side of the equation is simplified by the omission of other terms.

End conditions are that the end moments are zero except when there is a load on an overhung end, in which case the actual value of this end moment can be inserted.

Overhung ends are invariably shorter than the main spans, but for construction of influence lines they may be taken of the same length, and only the required range of k used.

Some doubt may be felt as to the applicability of the five moment equation above when there are less than five supports, or for end spans. Since however the terms in M_D and M_E only appear as a result of substituting for the reactions R_B and R_D, correct results are obtained by putting $M_A = 0$ when B is the first support and $M_E = 0$ when D is the last support.

In the case of an overhung end which is considered as the first span, $M_A = 0$ and the term $k_1 w_1$ on the right must be retained when the cantilever carries a load. M_B then has the value $(1 - k_1)\, W_1 l$.

Solutions for Three Supports.—Referring to Fig. 24. Load on 1st span:—$M_B = (1 - k_1)\, W_1 l$, $M_A = 0$, $M_D = 0$, $M_E = 0$.

The elastic equation is :—

$$(1 - 4x)(1 - k_1)\, W_1 l + (4 + 6x)\, M_C = x k_1 W_1 l$$

$$\text{giving } M_C = \frac{k_1 - 1 + x(4 - 3k_1)}{4 + 6x}\, W_1 l$$

Load on 2nd span :— $M_A = M_B = M_D = M_E = 0$.

The elastic equation is :—

$$(4 + 6x)\, M_C = W_2 l\, \{ K_{R_2} + x(1 - 3k_2) \}$$

$$\text{giving } M_C = \frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x}\, W_2 l$$

These results are plotted as influence lines for $x = 0$, $x = 0.5$ and $x = 1.0$ in Fig. 25.

In general the reactions are best deduced from the moments by means of the expression already given. In this case the values reduce to the following.

With the load on the overhung span AB :—

$$\frac{R_B}{W_1} = (2 - k_1) + \frac{(1 - k_1) - x(4 - 3k_1)}{4 + 6x}$$

$$\frac{R_C}{W_1} = \frac{x - 3(1 - k_1)}{2 + 3x}$$

$$\frac{R_D}{W_1} = \frac{(1 - k_1) - x(4 - 3k_1)}{4 + 6x}$$

With the load on the second span :—

$$\frac{R_B}{W_2} = (1 - k_2) - \frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x}$$

$$\frac{R_C}{W_2} = \frac{K_{R_2} - 2k_2 + x}{2 + 3x}$$

$$\frac{R_D}{W_2} = -\frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x}$$

Solution for Four Supports.—Referring to Fig. 26, two five moment equations are required in this case, together with the end conditions.

Loads W_1 W_2 W_3 only need be considered.

$$M_A = M_E = M_F = 0, \quad M_B = (1 - k_1)\, W_1 l.$$

The elastic equations are then :—

$$(1 - 4x)(1 - k_1)\, W_1 l + (4 + 6x)\, M_C + (1 - 4x)\, M_D = K_{R_2} W_2 l + K_{L_3} W_3 l + xl\, \{ k_1 W_1 + (1 - 3k_2)\, W_2 + (3k_3 - 2)\, W_3 \}$$

$$x(1 - k_1)\, W_1 l + (1 - 4x)\, M_C + (4 + 6x)\, M_D = K_{R_3} W_3 l + xl\, \{ k_2 W_2 + (1 - 3k_3)\, W_3 \}$$

The solutions for M_C and M_D are, for load on 1st span, the overhung end only :—

$$\frac{M_C}{W_1 l} = \frac{(5x + 4)(4x - 1) - k_1(14x^2 + 7x - 4)}{(2x - 5)(10x + 3)}$$

$$\frac{M_D}{W_1 l} = \frac{(10x^2 - 12x + 1) - k_1(6x^2 - 11x + 1)}{(2x + 5)(10x + 3)}$$

For load on second span BC :—

$$\frac{M_C}{W_2 l} = \frac{(K_{R_2} + x)(6x + 4) - k_2 x(14x + 13)}{(2x + 5)(10x + 3)}$$

$$\frac{M_D}{W_2 l} = \frac{(K_{R_2} + x)(4x - 1) - k_2 x(6x - 7)}{(2x + 5)(10x + 3)}$$

For load on third span CD :—

$$\frac{M_B}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1) + (K_{L_3} - 2x)(4 + 6x) + 3k_3 x(2x + 5)}{(2x + 5)(10x + 3)}$$

$$\frac{M_C}{W_3 l} = \frac{(K_{R_3} + x)(6x + 4) + (K_{L_3} - 2x)(4x - 1) - 3k_3 x(2x + 5)}{(2x + 5)(10x + 3)}$$

General expressions for reactions will not be given in this case. They can be deduced as in the case of three supports.

Solution for Five Supports.—Proceeding in the same way three equations can be written down and solved for M_C, M_D and M_E. The results are as follows :—

$$\frac{M_C}{W_1 l} = \frac{(4x - 1)(15x^2 + 52x + 15) - k_1(40x^3 + 137x^2 - 7x - 15)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_1 l} = \frac{(9x^2 - 12x + 1) - k_1(5x^2 - 11x + 1)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_1 l} = \frac{(4x - 1)(5x^2 - 16x + 1) - k_1(10x^3 - 57x^2 + 19x - 1)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For load on second span BC :—

$$\frac{M_C}{W_2 l} = \frac{(K_{R_2} + x)(2x + 5)(10x + 3) - k_2 x(40x^2 + 157x + 49)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_2 l} = \frac{(K_{R_2} + x)(4x - 1) - k_2 x(5x - 7)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_2 l} = \frac{(K_{R_2} + x)(10x^2 - 12x + 1) - k_2 x(10x^2 - 47x + 7)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For load on third span CD :—

$$\frac{M_C}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1)(5x + 4)}{2(5x + 4)(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(2x + 5)(10x + 3) + k_3 x(10x^2 + 123x + 58)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_3 l} = \frac{(K_{R_3} + x)(7x + 4)}{2(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(4x - 1) - k_3 x(5x + 16)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1)(5x + 4)}{2(5x + 4)(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(10x^2 - 12x + 1) - k_3 x(10x^2 + 13x - 30)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For more than five supports, involving more than three simultaneous equations, solutions in terms of x, the stiffness ratio, become so clumsy that it is better not to set them out but to solve the equations directly with numerical coefficients for particular values of the stiffness ratio as required.

Solutions for Uniformly Distributed Dead Loads.—If the beam has cantilever ends, it is of course necessary to know their length. It will be assumed in the solutions given below that the length of the cantilevers is half the length of the interior spans.

Case of Three Supports.—Using the general equation, $M_A = M_E = 0$, $M_B = M_D = \frac{wl^2}{8}$, $W_1 = \frac{wl}{2} = W_4$, $W_2 = wl = W_3$, $k_1 = \frac{3}{4}$, $k_2 = k_3 = \frac{1}{2}$, $k_4 = \frac{1}{4}$, $K_{R_2} = K_{L_3} = \frac{1}{4}$.

Substituting these values :—

$$(1 - 4x)\frac{wl^2}{4} + (4 + 6x)\, M_C = \frac{wl^2}{2} + xwl^2 \{ \tfrac{3}{8} - \tfrac{1}{8} - \tfrac{1}{2} - \tfrac{3}{8} \}$$

giving

$$M_C = \frac{3x + 1}{3x + 2} \cdot \frac{wl^2}{8}$$

Case of Four Supports.—Similar treatment gives

$$M_B = M_E = \frac{wl^2}{8}$$

$$\text{and } M_C = M_D = \frac{2x + 3}{2x + 5} \cdot \frac{wl^2}{8}$$

Case of Five Supports.—$M_B = M_F = \frac{wl^2}{8}$

$$M_C = M_E = \frac{5x^2 + 24x + 4}{5x^2 + 34x + 7} \cdot \frac{wl^2}{8}$$

$$M_D = \frac{5x^2 + 19x + 5}{5x^2 + 34x + 7} \cdot \frac{wl^2}{8}.$$

VINCENTS RIVULET BRIDGE

PLAQUING CEREMONY REPORT

In order to hold the ceremony on site, it would have been necessary to tidy up the bridge and its environs and to erect a marquee in which refreshments could be served. It was decided instead to have an evening function at the Woodstock Reception Centre in Hobart.

As the Governor had a special interest in the bridge and agreed to unveil the plaque, a mutually convenient date was set, and the ceremony was planned for 4th November 1999.

Invitations were sent to current and retired bridge engineers, Allan Knight's family and office bearers in the IE Aust Tasmania Division, the National Committee on Engineering Heritage and the Royal Society of Tasmania. In addition the event was advertised in Engineers Tasmania and in the Royal Society circular, and any members who expressed an interest in attending were sent an invitation. Catering was based on the acceptances received.

The function was well supported by the Royal Society of which the Governor is the President, and the total attendance was about 70.

Programs placed on the seats showed a photograph of Sir Allan Knight standing beside the bridge, and included information about the bridge, the plaquing program and the order of proceedings. Our regular MC Keith Drewitt was suffering from a virus, and Bruce Cole filled the void.

All the speakers spoke well and for the appropriate length of time. Mr Ivan Gaggin described the events leading up to the design and construction of the bridge in 1932 and Sir Allan Knight's major role. Dr Steve Carter presented the plaque on behalf of the IE Aust. The Governor spoke before unveiling the plaque. Mr Mark Addis accepted the plaque, as his agency is the current owner of the bridge. He undertook to arrange for the plaque to be erected near the bridge in due course. Photographs were taken by Heritage Committee members Allen Wilson and Fred Lakin.

After the ceremony the guests stayed on for a chat. Supper provided by the agency and drinks by the IEAust. There were no glitches and overall the function was very successful.

The Royal Society contributed $200 towards the cost of the plaque and the Society's name and logo appears on it.

Copies of the invitation, program, photographs and one speech are attached to this report.

Bruce Cole

The Institution of Engineers, Australia
and
The Royal Society of Tasmania

The President of the Tasmania Division of the Institution of Engineers, Australia,
and the
Vice-President of the Royal Society of Tasmania
cordially invite

---

to attend a ceremony on
Thursday 4th November 1999 at 7.30 pm
to commemorate the

VINCENTS RIVULET BRIDGE

with the unveiling of an

Historic Engineering Marker

by the Governor of Tasmania
Sir Guy Green AC, KBE.

The ceremony will be held at the
Woodstock Reception Centre,
140 Cascade Road, South Hobart.

Refreshments will be served after the ceremony.

RSVP
by Wednesday 20th October 1999
Tel: (03) 6234 2228

Attendees are requested to be seated by 7.20pm

The INSTITUTION of ENGINEERS, AUSTRALIA
and
The ROYAL SOCIETY of TASMANIA

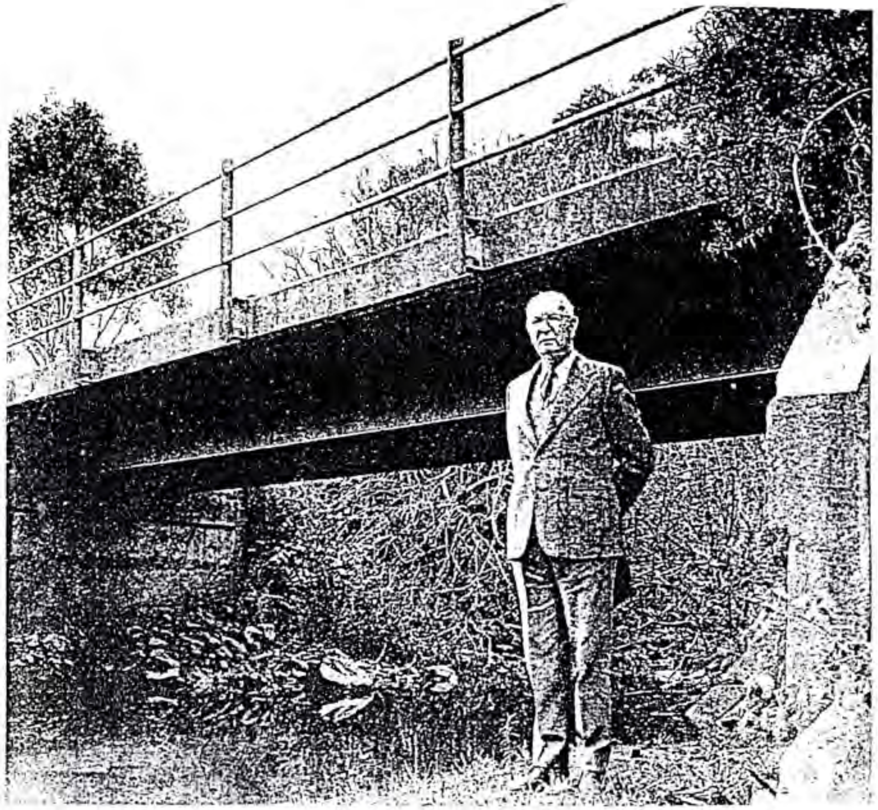
Official Ceremony
for the unveiling of an

HISTORIC ENGINEERING MARKER

for the

VINCENTS RIVULET BRIDGE

on 4th November 1999



Vincents Rivulet Bridge and Sir Allan Knight

A Brief History

In the early 1930s, (Sir) Allan Knight was working as a demonstrator in the Engineering Department of the University of Tasmania under the direction of Professor Alan Burn.

Whilst doing a bridge research project for the Public Works Department, Allan Knight developed the theory that, if a concrete deck could be made to act with the supporting steel beams without slippage or separation (called composite action), the same structure would have greater strength and carry a higher load.

When Mr Knight joined the PWD in 1932, the Director Mr G D Balsille authorised building a small bridge to test the effect of the composite bridge theory.

The bridge was built on Proctors Road over Vincents Rivulet. Composite action was achieved by welding steel stirrups to the beams at 7 inch (175 mm) centres. The concrete deck was then poured and, after the time had elapsed for the concrete to gain full strength, a 10½ ton test load was applied.

Deflection of the bridge was negligible, proving the theory which was then applied in building larger bridges with considerable financial savings.

Australian Engineering Plaquing Programme

The erection of plaques attracts public attention to worthy historic engineering works and sites. A national committee awards a plaque only after the preparation of a detailed submission and approval. National icons which have received plaques include the Sydney Harbour Bridge, the Goldfields Water Supply Scheme in Western Australia, the Snowy Mountains Scheme and our Waddamana 'A' Power Station. Many more works of state significance have been plaqued.

Here in Tasmania plaques have been presented for the Richmond Bridge, Kings Bridge in Launceston, Waddamana as mentioned above, the McNaught Beam Engine (on display outside the TAFE College in Hobart) and the Tarraleah Power Development. The Vincents Rivulet Bridge plaque is the sixth to be presented in Tasmania.

PLAQUING CEREMONY PROGRAMME

held at Woodstock, Hobart

Master of Ceremonies

Mr Bruce Cole, FIEAust
Secretary, Tasmania Division Engineering Heritage Committee

Historical Introduction

Mr Ivan Gaggin, FIEAust
Former PWD Bridge Engineer.

Presentation

Dr Steve Carter, MIEAust
President, Tasmania Division, Institution of Engineers, Australia.

Unveiling of Historic Engineering Marker

His Excellency Sir Guy Green, AC, KBE
The Governor of Tasmania

Response

Mr Mark Addis
Secretary, Dept of Infrastructure, Energy & Resources.

Conclusion

Mr Bruce Cole, FIEAust

VINCENT'S RIVULET BRIDGE

Historical Introduction

IVAN GAGGIN, BE

It is not very often that bridges of rare distinction come the way of bridge engineers. I have been fortunate to have had a couple, and they have added great additional interest to what is already an interesting field of work. One of these rare bridges was Vincent's Rivulet Bridge on Proctors Road in its descent from Tolman's Hill to Kingston - a small single lane concrete bridge, with steel beams, spanning but 10.4 metres, a deceptively inconspicuous bridge.

The rather light steel fence had been heavily damaged by vehicle impact so we repaired the fence, fixed up the pot-holed approach road, and installed guard rails leading up to the bridge. In the process, I learned that Vincent's Rivulet Bridge was a very early composite-action bridge. By composite-action is meant that the concrete slab is, in some way, structurally connected to the steel beams so that the two act together in the spanning action between substructures. Concrete deck slabs prior to composite action supported the vehicle vertically and spanned transversely between steel beams delivering the vehicle loads to the steel beams. Composite action used the concrete in the one remaining direction - longitudinally with the girder. Great economies with this new form of construction could be achieved.

At an earlier State Bridge Engineers Conference I attended, I had heard a claim by one of the members about a composite action bridge in his State being the first in Australia. Coming across Vincent's Rivulet Bridge gave me a much earlier date for composite action. I decided to speak to the designer and construction supervisor of the bridge, who at the time, was a young graduate engineer from the University of Tasmania, Mr Allan Knight. So I rang Sir Allan and asked if he would talk to me about Vincent's Rivulet Bridge. He readily agreed. His story was as follows:

In 1932 Australia was in the depths of the Great Depression. Mr Knight was working as a part-time demonstrator in the Engineering Department, University of Tasmania, under the direction of Professor Alan Burn. Mr George Balsille, Director of the Public Works Department, approached Professor Burn in early 1932 to see whether he could assist in determining for what percentage of the road vehicle weight each bridge beam should be designed. By accurately knowing this percentage the design would be more certain, and savings in material and therefore money may be made. Professor Burn accepted the commission, and asked Mr Knight if he would undertake the work. Mr Knight thought that the best way to investigate the problem was to build a structural model and apply known point loads at various positions on the deck and to measure these deflections by means of an extensometer, while at the same time develop a mathematical theory.

He used a bakelite deck slab placed on light timber laths, and applied known point loads to the deck and measured the deflections of the laths at various span positions. He was initially disappointed that the deflections measured did not tally with the mathematical theory he had produced. However, in doing further loadings and measurements, he noticed that, on inspecting the underside of the model, the bakelite deck had actually lifted off and parted company with some of the supporting beams. He realised that separation did not take place on actual structures; so to overcome this difficulty and ensure that separation did not occur, he melted paraffin wax along the top of the timber laths and stuck the bakelite deck to the laths with this wax. He then resumed testing of the model and noticed two things had changed:

(i) a fairly good correlation with the mathematical theory was achieved, and
(ii) the deflections of the timber laths were now much smaller than previously indicated, and by gluing the deck to the laths he had produced a much stiffer bridge deck.

He was able to supply the Public Works Department with the information they sought, but was also able to conclude that if a concrete deck could be made to act with the supporting steel beams without slippage or separation, economies could be afforded in bridge decks for given loadings.

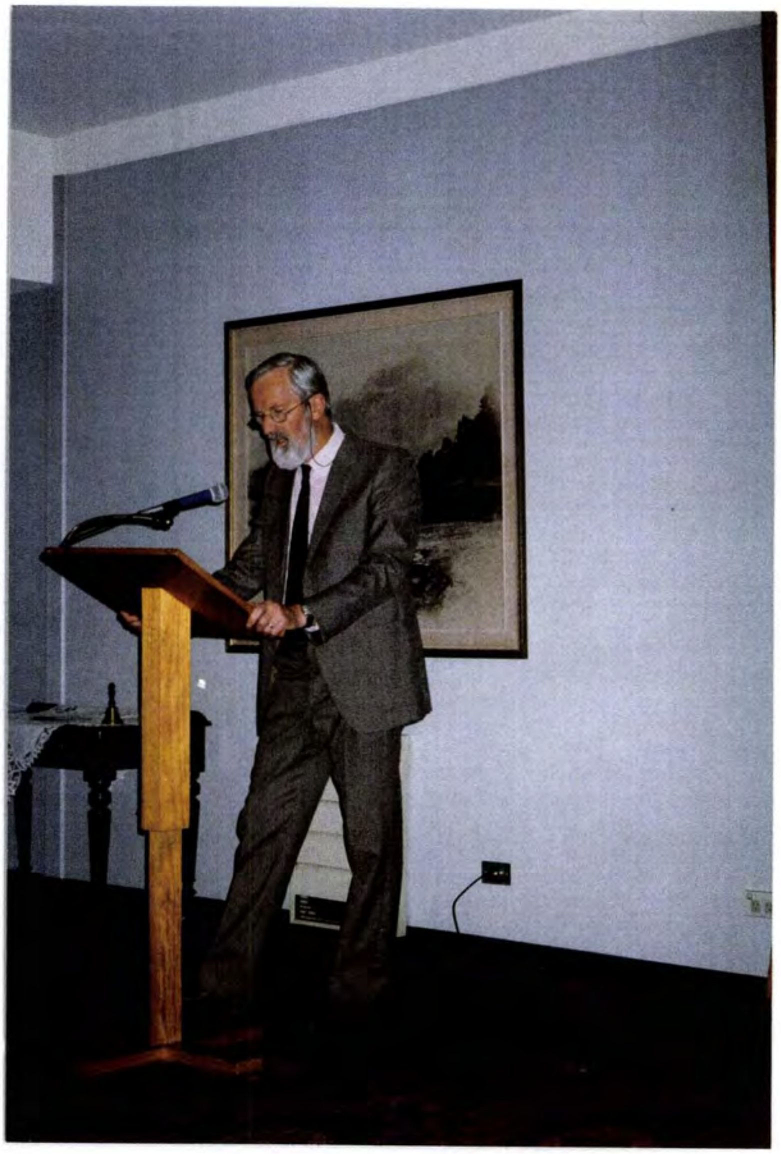
And there the matter may have rested. But in September 1932 Mr Knight joined the staff of the Public Works Department in their Bridge Section and proposed that the Department adopt the principle of making the concrete deck act compositely with the supporting steel beams. Mr Balsille, the Director, was interested in promoting the proposal, and consideration was given to building a model in the railway yard, but at the last minute the site was taken for something else. A 1/6 scale single steel beam was given a concrete slab with composite action being achieved by welding round steel shear reinforcement to the steel beams and turning them up into the concrete deck. This model beam was tested and produced deflections that were closely in accord with structural theory without slippage or separation. So the decision was made to proceed and build a full scale bridge to give the system a full scale test. True caution prevailed, and a quiet back road was selected so that if unforseen problems arose no serious dislocation of traffic would occur. And so Vincent's Rivulet Bridge on Proctor's Road was build, and did, indeed, incorporate further advances in composite action design by local deepening of the concrete deck above each steel beam to provide a deeper stronger section, and temporary propping of the steel beams so that composite action was provided for dead load as well as live load. The bridge, upon completion, was test loaded with a 10.6 ton truck loaded with road metal and came through with flying colours. Messrs Balsille and Knight were in the group that had gathered to witness the test loading, and after its completion, left the site to celebrate their success in an appropriate manner.

The success of Vincent's Rivulet Bridge, in demonstrating the validity and value of composite action between the concrete deck and supporting steel beams, encouraged the further and wider use of composite action in Departmental bridgeworks, the next and larger bridge being the Leven River Bridge at Ulverstone. Mr Knight published a Paper on composite action in "The Commonwealth Engineer" (April 1st 1933 issue) and awakened wide interest throughout Australia, with other States adopting the principle and constructing bridgeworks incorporating composite action.

After my interview with Sir Allan Knight, I wrote a Technical Note for The Institution of Engineers, Australia and in that Note made the statement that; "As far as can be determined, this bridge is the first composite concrete deck/steel girder bridge built in Australia, and perhaps the world." I cleared the Note with Sir Allan, and it was subsequently published, and no comment was received to contradict the statement.

When the time came for constructing the second carriageway of Hobart's Southern Outlet Road, the alternatives of placing the new carriageway on the eastern or western side of the first carriageway were explored. The eastern alternative would have meant the covering of Vincent's Rivulet which runs parallel to the road for a considerable distance and, of course, the obliteration of Vincent's Rivulet Bridge. The western alternative involved some heavy cutting into the rising country. The existence and importance of the bridge was made known to the investigators. Fortunately, the cost of a very long length of concrete culvert for the rivulet outweighed the cost of the earthworks and the western alternative was adopted, leaving the Bridge in peace.

Vincent's Rivulet Bridge continues to enjoy its quiet life on Proctors Road, immediately adjacent to Hobart's busy outbound Southern Outlet Road. It can be viewed from the Outlet Road, but is best inspected from Proctors Road itself. The bridge will continue to enjoy a high place of honour amongst the many bridges in the State of Tasmania, and it is hoped will be maintained in first class condition in keeping with its historic nature and value.



Bruce Cole MC

Ivan Gaggin presented the history of the Bridge.

Steve Carter, President of the Tasmania Division, IEAust,
presented the plaque.

H E The Governor of Tasmania, Sir Guy Green,
President of the Royal Society of Tasmania,
spoke before unveilling the plaque.

Mark Addis, Secretary
Dept of Infrastructure, Energy & Resources
accepted the plaque on behalf of the owner.

The Governor unveiled the plaque,
assisted by Bruce Cole, Master of Ceremonies.

Vincents Rivulet Bridge Historic Engineering Marker